Sierra Scientific Services

An Evaluation of Well Placements and Potential Impacts of the ID4 / Kern Tulare / Rosedale - Rio Bravo Aquifer Storage and Recovery Project, Bakersfield, California.

20 July, 2004

prepared for:

Kern County Water Agency Improvement District No. 4
P. O. Box 58
3200 Rio Mirada Dr.
Bakersfield, CA 93302-0058
Attn: Mr. Martin Varga, Project Engineer
(661) 634 - 1448

prepared by:

Sierra Scientific Services
2609 Highland Ct.
Bakersfield, CA 93306-2370
Attn: Robert A. Crewdson, Ph.D.
(661) 872-4221

Table of Contents

List of Figures (in Summary of Findings).

List of Figures (in Discussion).

List of Appendices.

1. Aquifer Geology and Hydrology.
2. Mathematical Aquifer Models.
3. Aquifer Parameter Values.
4. SSS Cooper - Jacob re-analysis of the ID4 December, 2002 Well Test.
5. KCWA report on the ID4 pump test of December, 2002.
6. Catalog of Drawdown Analyses for non- Base Case Conditions.
7. Limitations of the Analyses.

Sierra Scientific Services

An Evaluation of Well Placements and Potential Impacts of the
ID4 / Kern Tulare / Rosedale - Rio Bravo Aquifer Storage and Recovery Project.

1. Conclusions and Recommendations

The purpose of this Report is to present the findings of an impact evaluation for a well field of seven proposed wells which are a part of the ID4/KT/RRB aquifer storage and recovery (ASR) project. The projected recovery capacity of the well field is 90af/d (45 cfs) and the base case operating scenario is continuous pumping for 300 days per year in approximately three of every ten years years to produce 27,000 af/yr. The project wells are designed to be 1,000 ft or more away from the nearest non- project wells and 1,200 ft or more away from each other.

We conclude that the proposed well field minimizes drawdown impacts by putting the maximum available distances between wells. We conclude that for this project to operate as predicted and desired, the total recharge to this area must start out and remain in long term balance with total recovery in this area. We conclude that there is currently no recognized threat to water quality within the capture zone but it is very important to monitor the good water quality within well field capture zone for any contamination entering the flow paths leading to the wells.

This project represents only 25% of the total installed recovery capacity in the general area. We also observe, based on historical data, that the basinwide water level response to the climatic wet/dry cycle alone can be as large or larger than pumping drawdowns and may dominate the water level fluctuations in some years, independent of the project operations. Since project impacts may well occur at the same time as impacts from other causes, the combined year- to- year water level declines due to both climate and non- project pumping may be significantly greater than we have predicted due to project pumping alone.

For the base case scenario under leaky aquifer conditions, the predicted drawdowns within the aquifer during pumping will decline quickly and then stabilize at steady- state values of 16 - 24 ft at a distance of 1000 ft from the wells and 3 - 5 ft at a distance of 5000 ft from the wells. These drawdowns are as little as one- third of what would have been predicted under confined- aquifer conditions, as the aquifer has been modeled in the past by previous workers.

The project has considerable flexibility in delivering less than the full base case recovery volume of 27,000 af/yr. The project may meet reduced delivery obligations by choosing to pump for less time, and/or at lower pumping rates, and/or using fewer wells. Each of these possible alternatives provides different drawdowns and benefits, as we discuss in this Report.

Multi- year continuous pumping will not increase the drawdown as long as the project maintains its recharge commitment and the immediate area also continues to receive sufficient total recharge to re-supply all non-project wells in the area. The key to moderating the aquifer behavior is to keep the local area adequately recharged over time. If recharge does not match recovery, then the predicted drawdowns within the aquifer after 300 days of pumping will be as much as 60 - 70 ft at a distance of 1000 ft from the wells and 40 - 50 ft at a distance of 5000 ft from the wells.

For 300 days of pumping, the capture perimeter surrounding the entire well field extends only 500 - 1200 ft outward from the individual wells for this pumping period. For a hypothetical 30 years of continuous pumping, the capture zone would extend about 3,400 ft downgradient to the northwest and would extend a few thousand feet upgradient to the recharge boundary associated with the Kern river channel to the south and southeast.

Based on available literature, there are no known plumes or sources of contamination within the theoretical capture zone limit, but there are plumes of concern farther to the east. It is possible that, over time, these known contaminant plumes and any other unknown plumes in this area could be transported westerly by foreseeable aquifer dynamics to the point where they fall inside the capture limit of the well field. One important mitigation against the potential encroachment of contaminant plumes from the east is through the deliberate and sustained placement of local recharge to maintain the local ground water gradients at favorable levels and gradients.

We caution that the quantitative results of this entire study are based on a limited understanding of the aquifer and on a very small data set of existing, available, and verifiable parameter values which we have obtained from other sources. This ASR project presents an opportunity for the groundwater community to greatly benefit from the results of testing, monitoring, aquifer model calibration, and parameter verification that could be incorporated into this project. In our opinion, early and continued monitoring and verification will provide an important and useful baseline database in case the project has to defend itself against claims for impact damages. In some respects, this impact study is the first of its kind in this area, and the project operator has every opportunity to set the standard for good basin management within the program.

We recommend that the project test each new water well individually with a testing program which will provide for aquifer parameter measurement as well as pump parameter measurement. We recommend that the project partners consider contracting with SSS to help design, observe, and interpret the well tests.

We recommend that the project impacts be carefully monitored from startup so that we can calibrate and verify the results of this work program and then make refinements in our model of the aquifer behavior for future use.

We recommend installing monitoring wells to satisfy four different purposes, including well testing, model calibration and verification, long- term operational water level monitoring, and contaminant- detection monitoring. We recommend as many monitoring well installations as are necessary to cover all of these functions at all important locations. It may be necessary to install some monitoring wells which are useful for only one of these functions, since a single well placement may not be effective for all purposes. We recommend that the project consider designing the completion depth interval of each monitoring well depending on the intended purpose for the well. We also recommend that the project be willing to use multiple monitoring wells which are completed in different depth intervals where potentially effective or necessary.

We recommend that the project consider using the drawdown maps from this study to locate the placement of monitoring wells for water level monitoring especially in and around the recharge/recovery zones. We recommend that the project consider using the particle trajectory and capture zone maps from this study to locate the placement of monitoring wells for contaminant detection monitoring, especially to the east of the well field. We again recommend that the project consider restricting the completion depth interval of each monitoring well depending on the intended purpose for the well.

An Evaluation of Well Placements and Potential Impacts of the
ID4 / Kern Tulare / Rosedale - Rio Bravo Aquifer Storage and Recovery Project.

2. Summary of Findings

The purpose of this Report is to present the findings of a water well impact evaluation for a cluster of seven proposed wells which are a part of an aquifer storage and recovery (ASR) project. The technical elements of the work program included reviews of previous work, E-log evaluations, aquifer parameter determinations, computer modeling of water level drawdowns and aquifer flow trajectories, and interpretations. The figures which we reference in *this* section are at the back of this section.

Project Operations.

The proposed aquifer storage and recovery (ASR) project consists of conveyance facilities, recharge ponds, and seven proposed recovery wells (Location Maps, Figures A1 & A2). The proposed well field contains seven wells in an array which looks like a diamond-shaped kite with 4 wells at the corners and 3 wells in the tail. From tip to tail this array is about 5,500 ft long and is oriented west to east. The individual wells are about 1,200 - 1,500 ft apart. The base case operating scenario is to pump all seven wells for 300 days per year at a total flow capacity of 90 af/d (45 cfs) for an estimated 3 out of every 10 years. The most aggressive proposed scenario is to pump all wells at the full flow rate for 300 d/yr for 3 consecutive years. Less aggressive alternate operating scenarios include operating for fewer days, or at lower flow rates, or using fewer wells within a single operating year. Within this scope of work, we have analyzed the cases in which the project delivers only 50% and 25% of the full annual delivery of 27,000 af/yr.

The surrounding area is already being used by other entities for aquifer recharge, particularly along the Kern River channel, and for groundwater extraction by private domestic water supply wells and by municipal water supply wells. When completed, this proposed project will represent only 25% of the total groundwater extraction capacity in the immediate area, so the predicted impacts from this project represent only a fraction of the total possible local impact to the aquifer. The predicted impacts in this report refer only to the hypothetical impacts of operating this project.

Summary.

Based on our analysis, the actual aquifer behavior after pumping begins will depend significantly on the actual, but unpredictable, schedule and volume of the local recharge over the project life. If total local recharge remains in approximate balance with recovery (the balanced scenario), then the drawdown will decline quickly but stabilize at a shallow steady-state value. If local recharge fails to keep up with recovery (the unbalanced scenario), then the drawdown will decline in steps but will decline continuously as cumulative recovery dewaters the successive aquifer layers from top down.

In the balanced scenario, the capture zone will expand and water levels will decline for only 10 - 20 days before stabilizing at a predicted 16 - 24 ft drawdown at a distance of about 1,000 ft from the well field and about 3 - 5 ft drawdown or less at distances of 5,000 ft or more from the well field (Drawdown Maps, Figures A3 & A4). The steady- state condition of little or no water level decline may last for weeks or months and will continue as long as the extraction rate from the aquifer is fully supplied by leakage recharge from the overlying layers. This decline- and- stabilize behavior is characteristic of semi-confined, i.e., "leaky" aquifers. The predicted drawdowns which we calculated are based on leaky aquifer modeling.

In this scenario, the deeper, semi-confined aquifer will exhibit drawdown that is perhaps as little as one- third of what would be expected under confined conditions, as the aquifer has been modeled in the past. The price for this limited drawdown is that the shallow unconfined aquifer will be dewatered as it supplies recharge water to the underlying aquifer. The key to moderating this aquifer behavior is to keep the local area adequately recharged over time. If this happens, then the cyclic rise and fall of the water levels will remain within predicted ranges.

In the unbalanced scenario, the capture zone will expand and water levels will decline continuously, albeit at declining rates, but without ever stabilizing. This condition of declining water level will continue as long as pumping continues. This decline behavior is characteristic of unconfined aquifers which exhibit temporary flattening of the decline rate as delayed yield from overlying layers temporarily supplies some recharge to the extraction rate before continuing to dewater from the top down. The predicted drawdowns which we present for the alternate conditions described in this Report are based on unconfined aquifer modeling.

These predicted drawdowns are from this project alone under maximum recovery rates and do not include the potential impacts from other nearby pumping wells or the impacts due to basinwide water level changes due to the climatic wet/dry cycle.

Well placement.

The wells are optimally placed for minimum impact by spreading the well field over the largest available area and by placing the wells no closer than about 1,000 ft from nearby non-project wells. The nearest non- project wells (Well Location Map, Figure A2) are about 1,000 ft away and include 5 domestic wells, 3 City of Bakersfield wells, and 2 other municipal water supply wells. There are another 6 non- project wells out to 3,000 ft and another 13 non-project wells out to 5,000 ft away. Of these 29 wells within about a mile of the project perimeter, there are thirteen shallow domestic wells, seven deep City of Bakersfield wells, 4 deep municipal water supply wells, and five deep non- project ID4 wells. The predicted drawdowns at any of these locations may be read directly off the drawdown map (Figure A3).

Aquifer model.

The local aquifer beneath a depth of approximately 200+ ft is a semi-confined (leaky) aquifer separated from a shallow unconfined aquifer by a complex and heterogeneous aquitard which causes irregular vertical flux (Aquifer Cross Section, Figure A4). Based on E-logs, geologic cross sections, well test results and hydrograph behavior, we conclude that the sandy sedimentary layers below depths of 200 - 300 ft behave as a semi-confined aquifer which is separated from the overlying shallow, unconfined aquifer by an aquitard of interbedded silty and sandy sediments. For wells which are completed in the zones below the aquitard, the drawdown response to pumping is what hydrologist's refer to as "leaky" aquifer behavior, because recharge in response to pumping comes from the overlying layers as well as from aquifer storage within the zones of completion. SSS used the mathematics for leaky aquifer behavior to predict the drawdowns which we present in this Report. The leaky aquifer model is different than the confined aquifer model used by previous workers in other impact analyses on nearby projects.

Historical water level data shows that the natural ground water gradient changes over time. During wet years of the climatic wet/dry cycle, the gradient tends to point northwesterly in the project area under the influence of recharge in the nearby Kern River channel just south and east of the well field. During dry years of the climatic wet/dry cycle, the gradient tends to point due westerly in the project area under the influence of recharge and other dominating influences farther to the east. These gradient trends dominate the direction of the upgradient projection of the well field capture zone, as discussed in this Report.

Aquifer parameters.

Based on a review of published sample measurements and well test results, we have compiled a set of single values for each of the required parameters for our quantitative analysis. In our opinion, much of the available data are inconsistent and poorly documented, so that we are unable to corroborate or place measures of reliability on our parameter choices. The intrinsic parameter values we have used in our analysis are documented in this Report and include: Kh_{sand} = 80 ft/d, H_{sand} = 250 ft, Kv'_{silt} = 0.08 ft/d, H'_{silt} = 40 ft, S_s = 0.000041 ft^{-1}, S_y = 0.21, L' = 0.002 d^{-1}. We have also used values of T = 20,000 ft^2/d and S = 0.00056 which we re-calculated from the ID4 December, 2002 well test data. These parameter values are significantly different than the broad range of values reported by other workers in other impact analyses on nearby projects.

Base Case Drawdown.

For the balanced scenario under leaky aquifer conditions, the predicted drawdowns within the aquifer after 300 days of pumping would be 16 - 24 ft at a distance of 1000 ft from the wells and 3 - 5 ft at a distance of 5000 ft from the wells (Table of Drawdowns, Figure A6). The actual operating drawdowns will be similar to these calculated drawdowns if our selected parameter values are representative and if the true aquifer conditions at the time of startup reflect the assumptions of the balanced scenario.

For the unbalanced scenario under unconfined aquifer conditions, the predicted drawdowns within the aquifer after 300 days of pumping would be as much as 60 - 70 ft at a distance of 1000 ft from the wells and 40 - 50 ft at a distance of 5000 ft from the wells (Table of Drawdowns, Figure A5). The actual operating drawdowns will be similar to these calculated drawdowns if our selected parameter values are representative and if the true aquifer conditions at the time of startup reflect the assumptions of the unbalanced scenario, i.e., a lack of shallow recharge.

Alternate Case Drawdown.

For multi- year pumping under the balanced scenario, the leaky- aquifer model predicts that the steady- state final drawdown which is achieved after 10 - 20 days of pumping will remain steady indefinitely, even for 2 - 3 years, as long as the shallow aquifer layer is recharged and remains full of water. Without forecasting a specific future recharge schedule, it is impossible to determine if or when leakage will cease and the water table will continue to decline.

For multi- year pumping under the unbalanced scenario, the drawdowns are much bigger than the steady- state conditions, and the unconfined aquifer model predicts that the drawdown at any location after three years of pumping is only about 10% bigger than the unconfined drawdown after the first year of pumping. Thus, the gradual decline in the second and third years of pumping is relatively negligible compared to the majority of drawdown which occurs over the first year.

For ground water recovery of less than the full base case volume, the project has the following options in meeting the delivery obligation. These options provide choices which provide trade- offs between minimizing impacts and minimizing costs depending on future conditions and the potential relative impacts on the project and adjacent entities.

Option 1. To deliver 50% or 25% of the base case volume, the project may choose to pump all wells at the full rate for only 50% or 25% of base case the time, i.e., for 150 or 75 days instead of a full 300 days. The project would deliver 13,500 af and 6,750 af, respectively. The predicted drawdowns are the same as the full base case since both pumping durations exceed the 10 - 20 day period required to reach static decline. However, these water level drawdowns last only a half or a quarter as long.

Option 2. To deliver 50% or 25% of the base case volume, the project may choose to pump all wells for the full 300 days but at only 50% or 25% of the full design flow rate, i.e., at 45 or 22.5 af/d instead of the full 90 af/d . The project would deliver 13,500 af and 6,750 af, respectively. It is significant to note that the predicted drawdowns are only 50% and 25% as big, respectively, as the base case drawdowns, but they last the full 300 days. Of all the scenarios, reducing the total flow rate always makes the biggest reduction in the size of the drawdowns, all else being equal. Although pump efficiency is worse when pumps are not operated at their design flow rate, the value of secondary factors such as lifting cost should be carefully evaluated against the benefits of reduced drawdown impacts on the project and neighboring entities before this scenario is dismissed from consideration.

Option 3. To deliver about 50% of the base case volume, the project may choose to pump fewer wells. The project could operate just the five wells on the RRB property for the full 300 days at their full design flow rate of about 50 af/d. The project would deliver 15,000 af which is actually 56% of the base case delivery. The predicted drawdowns are slightly less than the base case and the capture zone covers a smaller area which is centered in the west half of the project area.

Alternately, the project may choose to pump just the two wells on the ID4 property for the full 300 days at their full design flow rate of about 40 af/d. The project would deliver 12,000 af which is actually 44% of the base case delivery. The predicted drawdowns are slightly less than the base case but slightly more than the RRB case, and the capture zone covers a similar area which is centered in the east half of the project area.

Option 4. To deliver about 25% of the base case volume, the project may choose to pump fewer wells at full rates and full durations or any combination of wells, rates, and durations which satisfy the delivery obligation. If the project operates just the two wells at opposite ends of the well field at full rates for the full duration of 300 days, then the drawdown would be spread over the largest possible area. The project would deliver 9,000 af which is actually 33% of the base case delivery. The predicted drawdowns are somewhat less than the base case but cover a comparable area. The expediency of this and other alternatives have not been evaluated or optimized within this scope of work with respect to secondary criteria.

Capture zone.

For 300 days of pumping, the capture zone of each well is small enough to be influenced by- but have only a little contact with- the capture zones of the adjacent project wells. The capture perimeter surrounding the well field extends only 500 - 1200 ft outward from the individual wells for this pumping period (300-day Capture Zone Map, Figure A7).

For a hypothetical 30-years of continuous pumping under a representative northwesterly groundwater gradient in an ideal aquifer, the capture zone would extend about 3,500 ft downgradient, extend about 9,900 ft upgradient, and cover an oval area of about 5100 ac (7.9 mi^2). In actuality, the hypothetical capture zone is smaller and the upgradient capture perimeter is actually much closer to the well field since the Kern River channel forms an upgradient recharge boundary only 3,000 - 6,000 ft away from the southern flank of the well field. The hypothetical capture perimeter for infinite pumping represents the theoretical capture zone limit for this well field, and the actual capture zones for shorter pumping periods will be smaller than, nested inside of, and conformal to this hypothetical capture limit (30-year Capture Zone Map, Figure A8).

Contaminant capture.

Contaminant transport is more complicated than groundwater flow because of the effects of dispersion, retardation, and attenuation along the flowpath. For this scope of work we assumed that a hypothetical molecule of contamination moves the same way as a molecule of water, so that any slug or plume of contamination within the capture zone limit will sooner or later reach the well field. And any contamination or potential source of contamination outside the capture zone limit will never reach the well field, except for one recognized case under the possible influence of changing conditions.

Based on the available literature, there are no known plumes or sources of contamination within the theoretical capture zone limit. To the north, the EDB and DBCP plumes which consultant Ken Schmidt identified near to- and north of- Rosedale highway, which is about 1.5 miles north of the well field, are outside the theoretical capture limit and therefore pose no threat to this well field. To the west, the capture zone does not extend very far downgradient to the west and northwest and there are no recognized sources of contamination in those directions. To the south, there are no known sources of contamination between the well field and the Kern River channel recharge boundary. To the east, there is no recognized contamination within the capture zone, but there are plumes of concern farther to the east which must be considered contaminants of concern.

Consultant Ken Schmidt identified several fuel- constituent plumes in the shallow aquifer near the oil refineries on either side of Coffee Rd, about 3 miles east of the well field. During wet years the potential migration pathway of these contaminants is to the northwest and along a trajectory which misses the capture zone of the project well field. The issue of concern is that during dry years when the gradient swings westerly, the potential migration pathway of these contaminants is directly toward the well field. Furthermore, based on KCWA groundwater maps for the area, the potential recharge of water in the Kern River channel just east of these plumes causes steep localized gradients which may accelerate the rate of movement of these plumes. In addition to the recharge gradients, non-project water wells to the east of the project well field will have capture zones of their own which may also create westerly gradients which draw these plumes toward those wells and, hence, also closer to this project's capture zone. It is possible that, over time, these known contaminant plumes and any other unknown plumes in this area could be transported westerly to the point where they fall inside the capture limit of the well field.

Key issues.

The number one key issue is that for this project to operate as predicted and desired, the total recharge to this area must start out and remain in long term balance with total recovery in this area. If the area remains balanced, then drawdown and contaminant capture impacts will remain within the predicted limits, subject to the identified uncertainties of this analysis. If the area becomes unbalanced, then drawdowns will worsen and contaminant capture dynamics may change significantly. The project itself is based on a program which is required to operate in balance but this project is only 25% of the identified recovery capacity in the area and the predicted recharge/recovery impacts, operating criteria, and water supply forecasts for these other wells are unknown.

The second key issue is that it is very important to protect the good water quality within the well field capture zone against any contamination entering the flow paths leading to the wells. There may be sources of contamination within the 30-year capture zone that we have no knowledge of. There are no operational safeguards that we know of to prevent contaminant capture if this is the case other than not pumping. The detection and delineation of unknown contaminant plumes doesn't lessen the seriousness of their eventual impacts unless the knowledge leads to mitigation or remediation. Such detection monitoring is beyond the scope of almost any affordable monitoring program unless there are abundant wells of opportunity upgradient of the well field that may be monitored in conjunction with dedicated monitoring wells which are installed in critical flowpaths (Individual- Well Capture Zones, Figure A9). One important mitigation against the potential encroachment of contaminant plumes from the east is through the deliberate and sustained placement of local recharge to maintain the local ground water gradients at favorable levels and gradients.

The third key issue is that this project represents only 25% of the total installed recovery capacity in the general area, which means that at any given time, some or all of an observed drawdown at some location could be caused by non-project pumping. Since project impacts may well occur at the same time as impacts from other sources, the combined drawdowns from project and non- project wells may be significantly greater than we have predicted due to project pumping alone. During climatic dry cycles, every well in the area may be pumping, and surrounding domestic wells may be significantly impacted. The cause- and- effect relationship between project and non- project wells and their proportionate share of the total impact cannot be easily resolved by direct observation alone. In our opinion, early and continued verification of the project impact model through well testing and drawdown monitoring will provide an important and useful baseline database in case the project has to defend itself against claims for impact damages.

The fourth key issue is that the dominant cause of water level fluctuations may be the basinwide response to the climatic wet/dry cycle. The rise and fall of the local water table due to the climate cycle is completely independent of- and may well be bigger in magnitude than- the combined impacts of local recharge and local pumping. For example, in the 20 years from 1984 - 2004 the water level in the project area has varied by more than 100 ft due to the impact of the climatic wet/dry cycle on the basin. In the decade from 1992 - 2002, the annual water level change due to non- pumping climatic factors was in the range of 20 - 30 ft in five different years. The project impacts and other local non- project impacts are superimposed on top of this broader, large- scale climatic trend. The generic cause and effect relationship between pumping and drawdown cannot be used to explain all future drawdowns without also considering the independent effects of basinwide behavior on the local area.

The fifth key issue is that the quantitative results of this entire study are based on a limited understanding of the aquifer and on a very small data set of existing, available, and verifiable parameter values. In our opinion, the uncertainty in the calculated drawdowns is not just due to the natural variability of the aquifer itself, but in the complete lack of verifiable replicate data apart from the single reported values which we used, which prevents us from even determining the range of actual values let alone estimating the uncertainty in these parameters.

In our opinion, the existence of this project and the likelihood of many more to come, point out the need to improve the quantitative understanding of the Kern Fan aquifer hydrology beyond the current rudimentary state of knowledge. This project presents an opportunity for the groundwater community to greatly benefit from the results of testing and monitoring that could be incorporated into this project. In some respects, this impact study is the first of its kind in this area, and the project operator has every opportunity to set the standard for good basin management within the program.

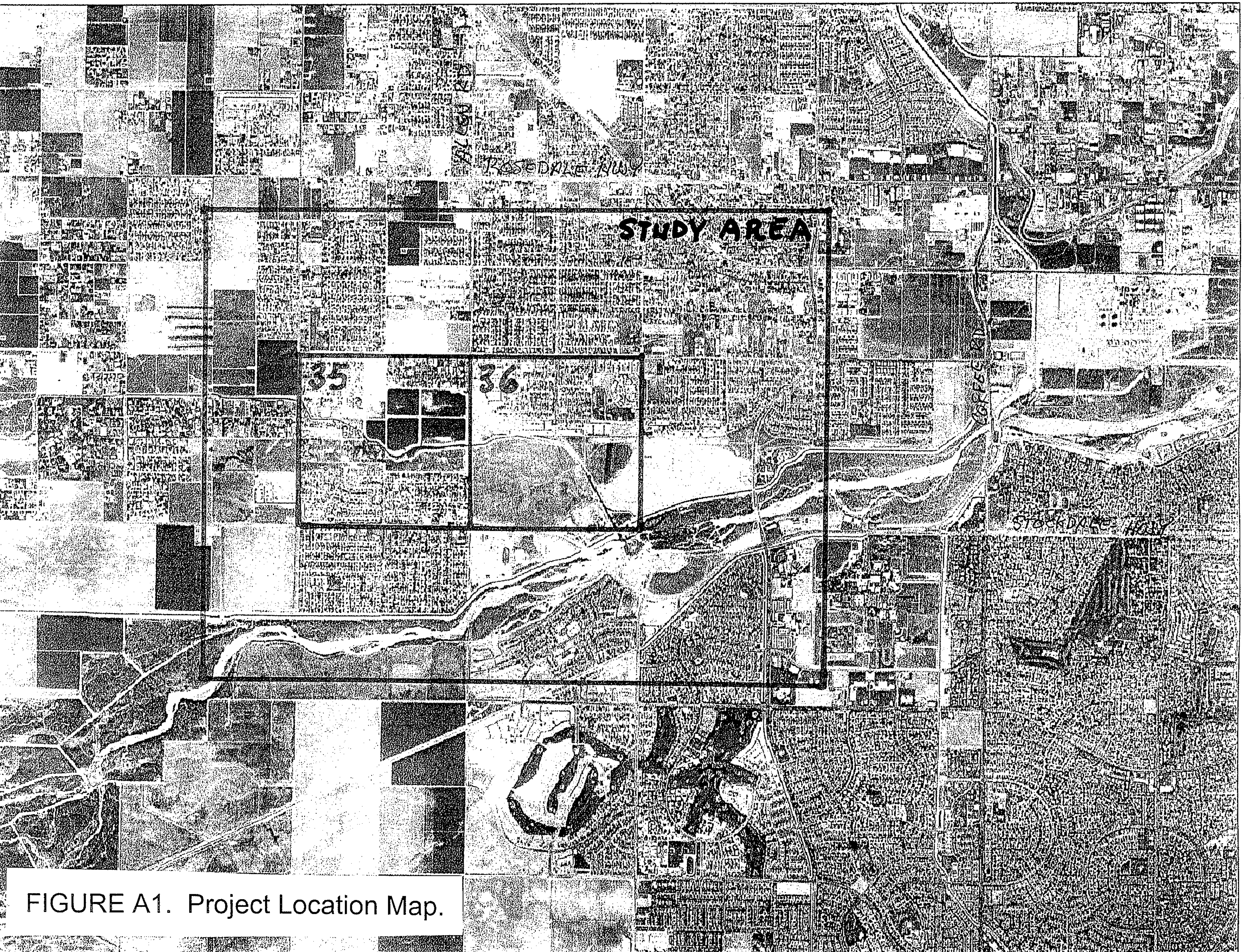


FIGURE A1. Project Location Map.

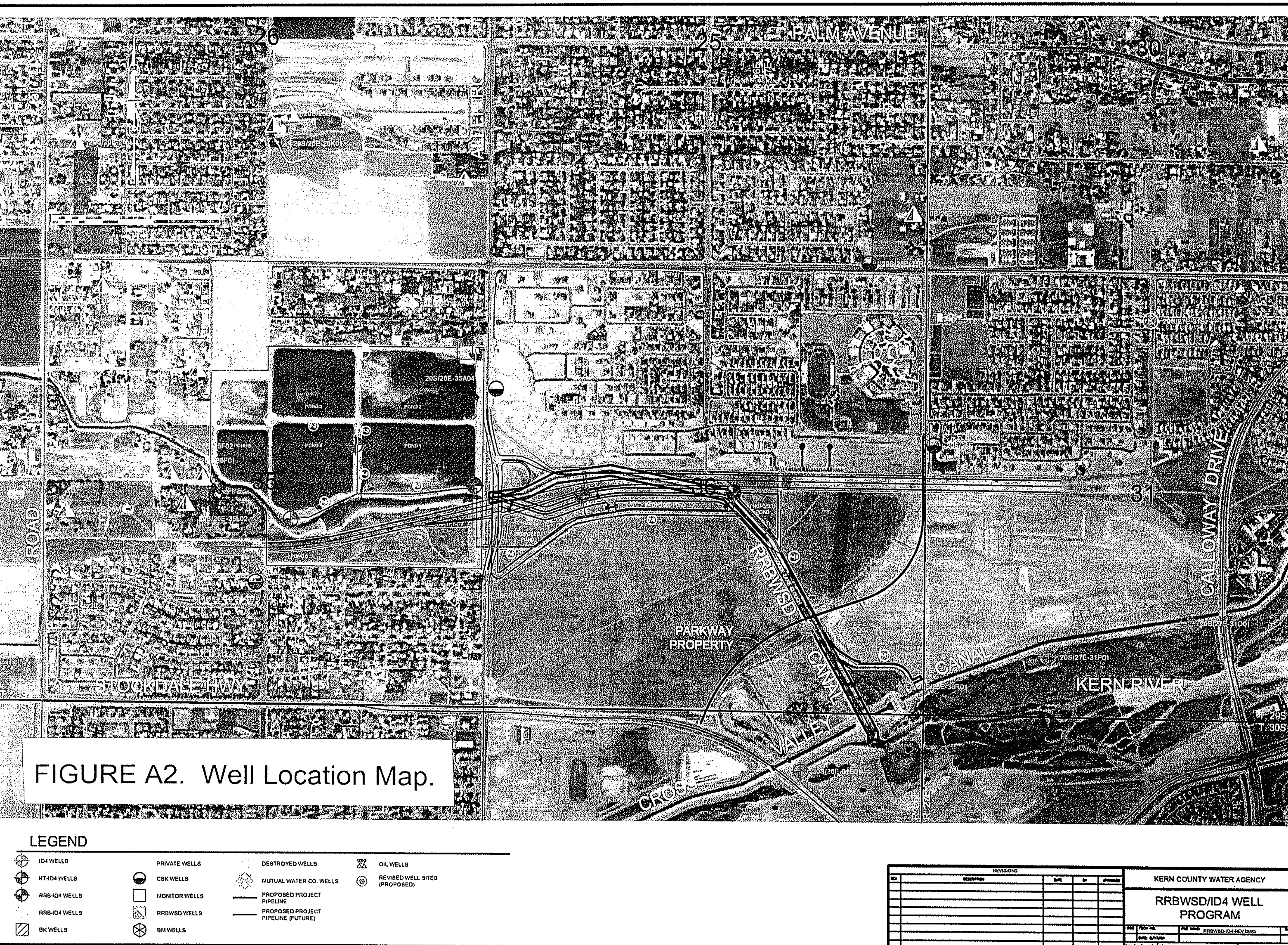


FIGURE A2. Well Location Map.

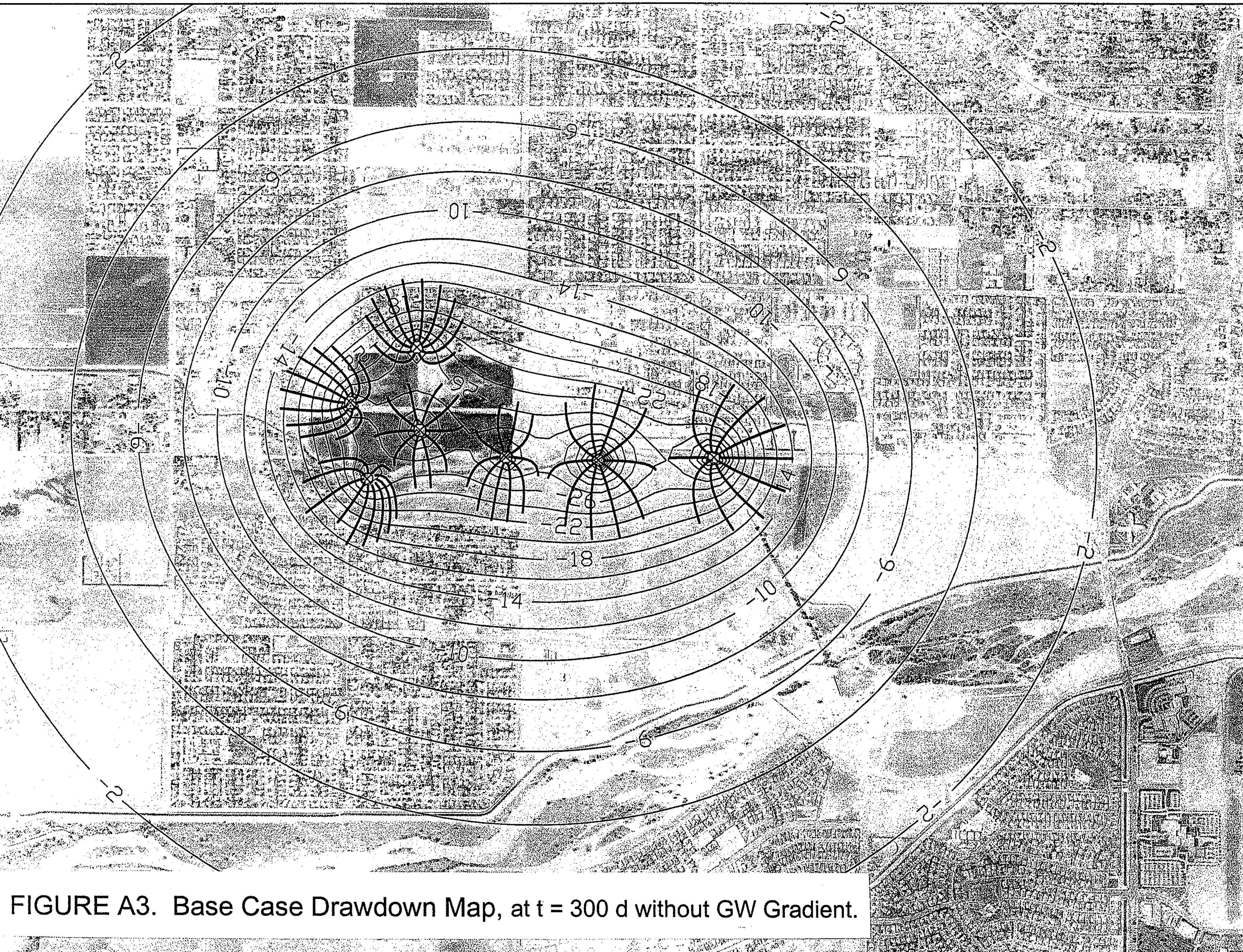


FIGURE A3. Base Case Drawdown Map, at t = 300 d without GW Gradient.

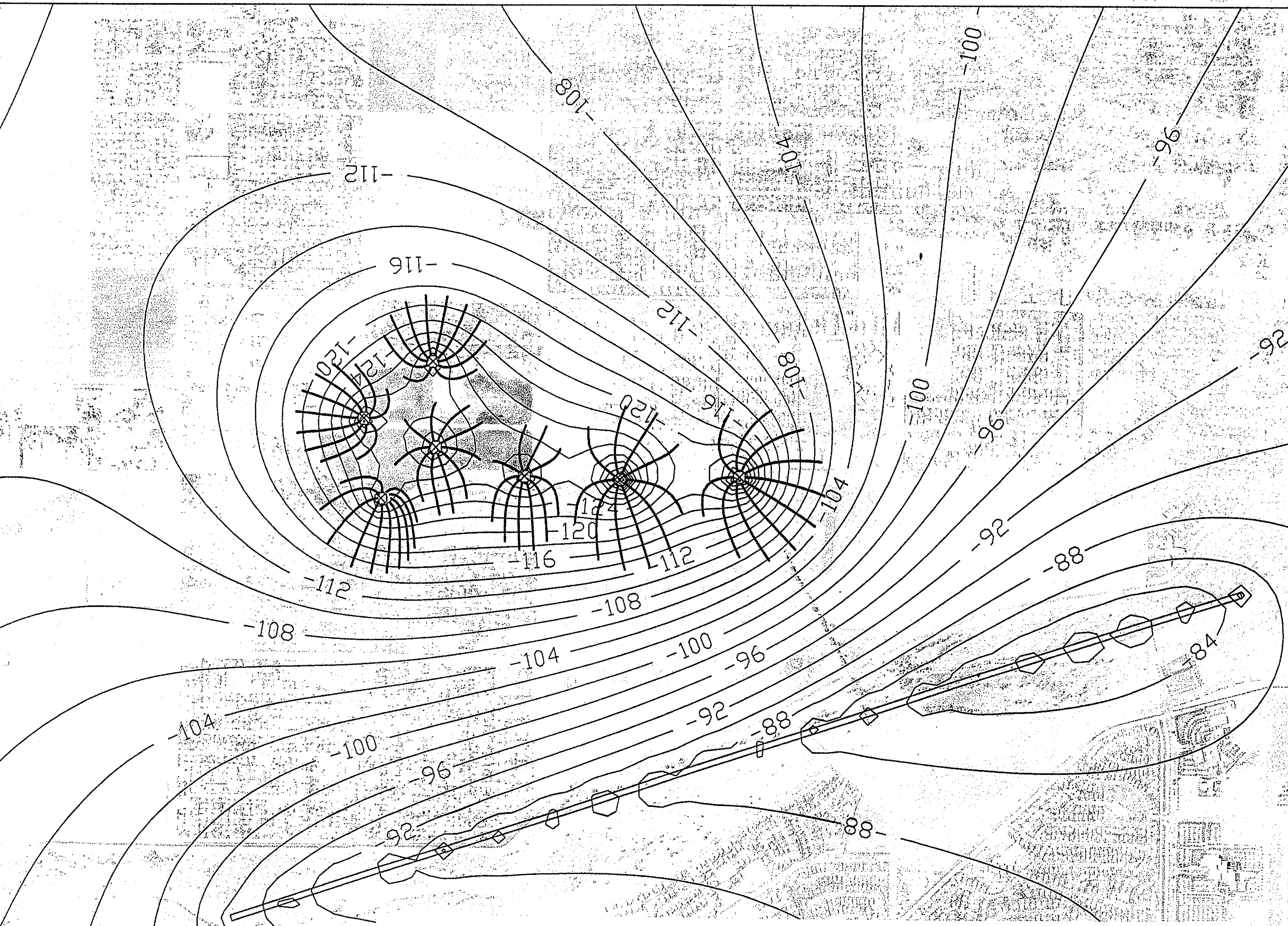


FIGURE A4. Base Case Drawdown Map, at t = 300 d with GW Gradient.

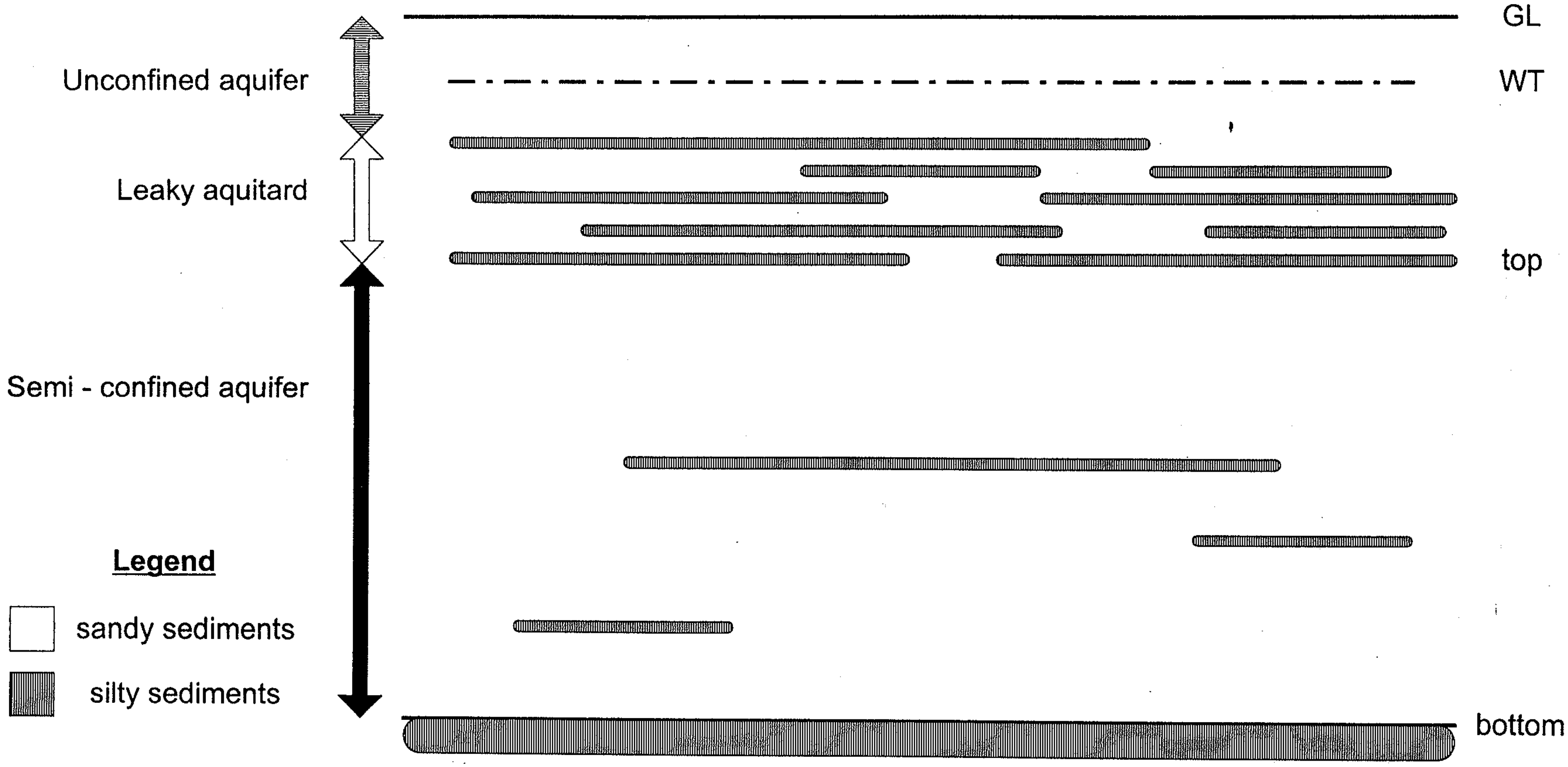


FIGURE A5. Schematic Geological Cross Section of a Semi- confined Aquifer.

ID4 / KT / RRB Well Field Predicted Drawdown Summary.

Aquifer Model		pumping time (d)	Predicted drawdown at 1000 ft (ft)	3000 ft (ft)	5000 ft (ft)
300 days pumping					
semi-confined, B = 2600		300 d	12 - 20	4 - 7	1 - 3
semi-confined, B = 3200	*	300 d	16 - 24	6 - 10	3 - 5
semi-confined, B = 6000		300 d	28 - 38	16 - 23	10 - 14
semi-confined, B = 10000		300 d	42 - 52	28 - 36	20 - 26
unconfined, low		300 d	22 - 30	13 - 19	9 - 14
unconfined, high		300 d	71 - 78	62 - 69	56 - 62
confined		300 d	120 - 132	103 - 115	98 - 103
3-years pumping					
semi-confined, B = 3200	*	3 yr	16 - 24	6 - 10	3 - 5
unconfined, high		3 yr	79 - 86	70 - 77	65 - 70
confined		3 yr	134 - 146	118 - 130	105 - 117

* = base case

FIGURE A6. Table of Calculated Drawdowns.

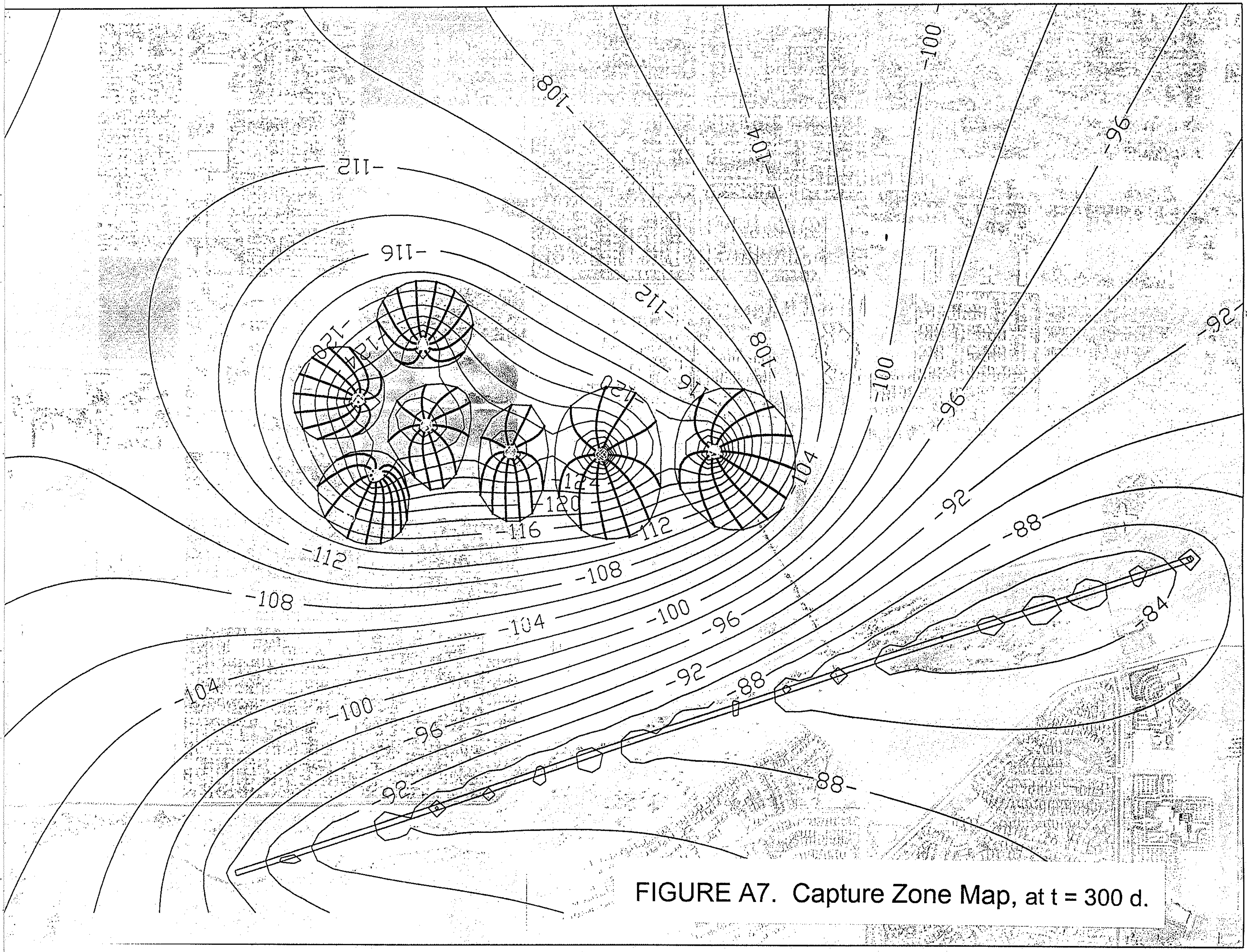


FIGURE A7. Capture Zone Map, at t = 300 d.

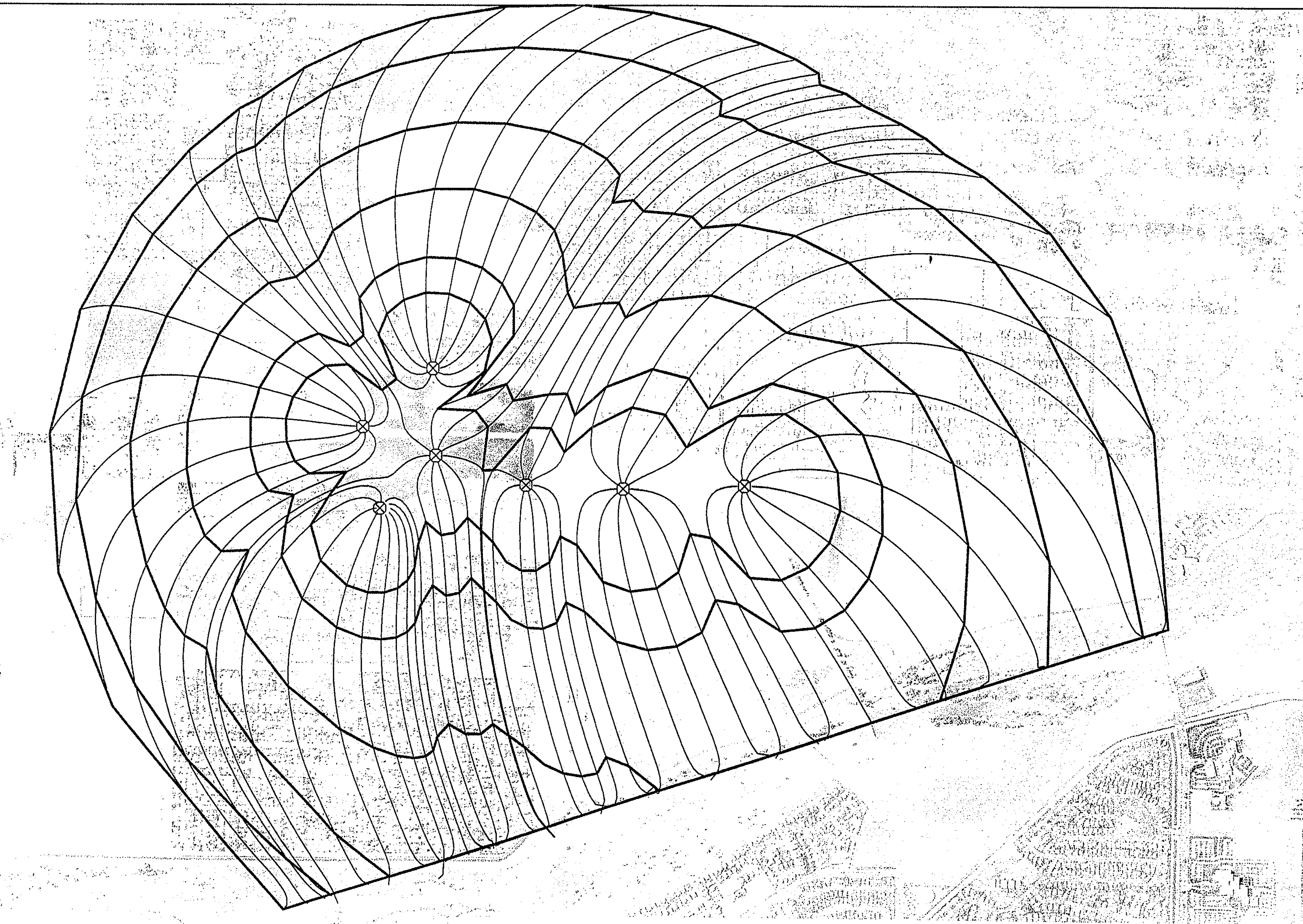


FIGURE A8. Particle Trajectory Map, for t = 1, 2, 5, 10, 20, 30 yr.

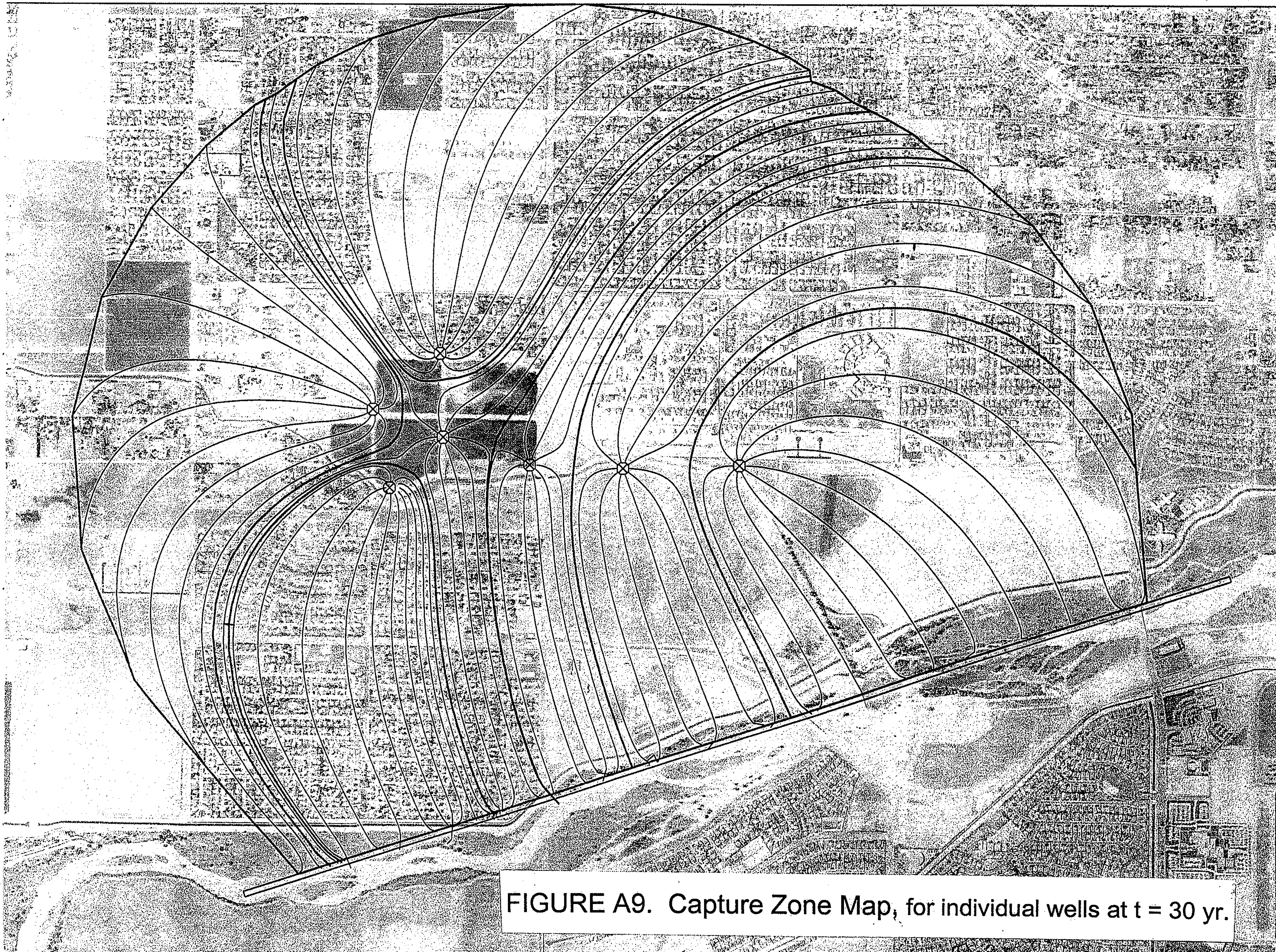


FIGURE A9. Capture Zone Map, for individual wells at t = 30 yr.

An Evaluation of Well Placements and Potential Impacts of the ID4 / Kern Tulare / Rosedale - Rio Bravo Aquifer Storage and Recovery Project.

3. Introduction

Section I - Work Program.

The focus of this work program is an array of seven proposed wells which are a part of the ID4/KT/RRB aquifer storage and recovery (ASR) project. The proposed scope of work includes a well placement and impact analysis including a review of surrounding wells, a geological review, an aquifer parameter selection, a pumping drawdown impact analysis for designated scenarios, a review of potential water quality impacts, and preparation of report.

The purpose of evaluating the well placements and impacts of the ASR Project is to demonstrate project viability within the proposed operating parameters to interested parties. The technical elements of the work program include E-log evaluations, aquifer parameter determinations, and computer modeling of water level drawdowns and aquifer flow trajectories.

Section II - Personnel.

Dr. Robert A. Crewdson is a Bakersfield, California consultant doing business as Sierra Scientific Services (SSS). SSS specializes in quantitative ground water hydrology, applied potential theory and time series analysis, quantitative ground water flow analysis, water quality geochemistry, well testing and monitoring, contaminant transport modeling, and aquifer properties testing. Dr. Crewdson is a research associate and adjunct professor at California State University Bakersfield where he teaches hydrology, contaminant transport, geochemistry and geophysics in upper division and graduate level courses.

Section III - Methods.

This Report presents the findings of several types of analyses and evaluations which we summarize here. The well log and geological analysis includes a review of the completion intervals in surrounding wells and a review of E-logs primarily for stratigraphy and net- sand analysis. The aquifer parameter analysis includes a review of published data, a review and re-calculation of T & S values from the December, 2002 ID4 well tests for nearby ID4 wells, and a review of published infiltration rate data for nearby recharge ponds and test ponds. The drawdown impact analysis includes Cooper- Jacob calculations for a range of transient conditions and parameters, and analytical computer modeling and mapping of multi-well drawdowns and particle trajectories. We used SSS proprietary software for the Cooper- Jacob calculations and WinFlow (tm) software by Environmental Simulations, Inc. for the drawdown modeling and mapping.

This impact analysis is the fourth such study that we know of in the general area of interest. The previous three impact analyses include reports on the Kern Water Bank (Schmidt, 1997), the Pioneer Project (Schmidt, 1998), and the ID4 Kern Parkway Project (Schmidt, 2003a & 2003b).

Section IV - Acknowledgments.

Thank you very much Tom Haslebacher, KCWA hydrogeologist, for providing the ID4 project location maps and for preparing and printing selected SSS data overlays on the KCWA air photo base map for inclusion in this Report.

4. Discussion

Section I - Project Description.

The ID4 / Kern Tulare / Rosedale - Rio Bravo Aquifer Storage and Recovery Project is a uniquely collaborative effort between three local water districts to operate and optimize an aquifer storage and recovery (ASR) project. The site of the Project is located about ½- mile north of the intersection of Allen Rd and Stockdale Hwy in southwest Bakersfield, Kern County, California (Figures 1 & 2).

The Kern Tulare Water District (KT) is the banking partner who will provide a wet- year water supply to the project and who will receive a dry- year water supply in return. The Rosedale - Rio Bravo Water Storage District (RRB) is the storage partner who will provide existing recharge ponds and aquifer storage. The Kern County Water Agency Improvement District No. 4 (ID4) is the operating partner who will engineer and operate the wells and pipeline facilities related to project operation. Kern Tulare WD and ID4 are funding the installation of five new water wells and a new pipeline gathering system.

Kern Tulare WD will store and recover an estimated net 243,000 acre-feet (af) of water over the initial 30 year project life. KT surface water will be delivered to the Project through turnouts or headworks that connect the RRB conveyance canal to both the nearby Cross Valley Canal (CVC) and the nearby Kern River channel. Well water will be returned to the CVC by the proposed pipeline gathering system. The forecast dry- year return of water is projected to occur in three years out of ten, and the well field operation is projected to be 90 af/d for 300 days in a return year.

The proposed project recovery design capacity is 90 af/d (approx. 45cfs) which includes three wells at 5 cfs and two wells at 10 cfs which will be installed in 2004 and another two proposed wells at 5 cfs which may be installed in the next 2 - 3 years. The full proposed well field straddles the boundary between RRB and ID4 with a 5-well array located on RRB property and a 2-well array located on ID4 property. The surrounding area contains private domestic water wells and municipal water wells of various depths which belong to private parties and other entities. Two of ID4's engineering design objectives are to 1. estimate the drawdown and water quality impacts of the proposed well field for a range of operating scenarios, and 2. minimize the drawdown impacts of the well field on surrounding wells by optimizing the well- location placements.

The proposed project ground water extraction capacity of 45 cfs is only 25% of the total estimated recovery capacity of 180 cfs for all of the wells in the local area. The City of Bakersfield owns 7 wells with an estimated pumping capacity of 45 cfs, there are 4 other recognized municipal water supply wells with an estimated capacity of 20 cfs, and 7 other ID4 wells with an estimated capacity of 70 cfs. It is reasonable to assume that during some dry periods when the demand for water is the highest, that all of these wells may be pumping at the same time. Our scope of work is limited to evaluating the potential impacts of this project operating as if it were operating alone.

Apart from selecting the proposed well locations, the drawdown impact analysis is the main objective of this evaluation. This analysis assumes that the wells are drilled, completed, and developed properly so that they are efficient and productive water wells, limited only by the delivery capacity of the aquifer. The drawdown impact analysis requires several types of essential information including operating parameters, well parameters, aquifer model and aquifer parameters. We describe each of these parameter sets below.

Section II - Project Operating Parameters.

For the purpose of impact analysis, the base case annual operating scenario is to operate all seven wells at specified flow rates for a combined design capacity of 90 af/d for 300 days, which produces 27,000 af of ground water over the period. The hypothetical long term project operation is based on the historical wet/dry climatic cycle which is predicted to require base case pumping in three years out of ten. The "worst-case" design scenario therefore is base case operation in three consecutive years. Other impact design scenarios of interest include one-year operations at 25% and 50% of base case.

There are, of course, many other possible operating scenarios and even multiple options in achieving the one- year operations at 25% and 50% of base case. These multiple options may include different pumping rates and/or different pumping durations for some wells or perhaps not using all of the available wells to meet the production target. It is outside the current scope of work to evaluate all of these possible operating scenarios, nor is it really necessary to do so. We have sensitivity analyses which cover the range of likely operating conditions.

There are other design variables which affect, and could perhaps even dominate, the impact analysis which are difficult to forecast in advance. The primary natural factors include the depth to the water table at project startup, the magnitude and direction of the ground water gradient, and the large basinwide water level fluctuation due to the climatic wet/dry cycle. The primary manmade variables include non-project impacts caused by other recharge or pumping operations in the surrounding area. The evaluation of these design variables is outside the scope of work.

Section III - Well Placement Analysis.

Several constraints and operating criteria limit the selection of the seven proposed project well locations. The two wells intended to be located on ID4 property needed to be placed along the proposed pipeline right of way and the five wells intended to be located on RRB property needed to be placed on the network of levees which provide access to the RRB recharge ponds. ID4 established the two locations on their property according to their own criteria so that the balance of the well placement analysis referred to the cluster of five wells on the RRB property.

The three main criteria for the five RRB well placements are to: 1. minimize well interference, 2. distribute the drawdown impacts as uniformly as possible across the largest possible area, and 3. minimize the drawdown impacts to any wells in the surrounding area. The first two criteria are best met by placing the wells on the nodes of a uniform rhombic grid at the largest possible spacing and operating all five wells simultaneously at the same flow rate. The third is best met by orienting and sizing the grid so that every possible well node is no closer to the nearest surrounding well of concern than a minimum specified standoff distance.

Five possible 5- well arrays (Figures 3A - 3F) fit the criteria for well placement and each array has a node spacing of 1200 ft between project wells and a minimum standoff distance of 1000 ft from surrounding wells. Each of the five possible well arrays has its advantages with respect to secondary criteria such as total gathering system pipeline length, lengths of larger and smaller diameter pipes, proximity to future road alignments, capital and operating costs, and other factors. Sierra Scientific used well array "A" for the calculation of hypothetical drawdowns (Figure 3A) and, for reference purposes, located each well on a coordinate system with respect to a local origin (0,0) at the intersection of Allen Rd and Stockdale Hwy.

Section IV - Aquifer Model and Parameter Selection.

There are many computation methods for predicting drawdown from a pumping well in space and time and every method requires that the user select the equations which are most appropriate for the user's preferred model of the aquifer. In essence, the user must try to select the set of mathematical expressions which best represent the user's physical model of the aquifer. The calculated results, if done correctly, always represent the mathematical model but only represent the real aquifer behavior to the extent that the parameters, simplifications and assumptions of the mathematical model reflect the true workings of nature. The selection of the mathematical model and the equations, the accuracy of the parameter values, and the representativeness of the calculated output all reflect the correctness of- and uncertainty in- the judgments of the user. These judgments cannot be made by the computer and the two main judgments include the choice of mathematical model and the choice of aquifer parameters.

<u>The Real Aquifer.</u> Based on our analysis of the E-log stratigraphy and hydrogeology, the local aquifer is a semi-confined (leaky) aquifer which is recharged from the sides and from the overlying layers. The aquifer consists of a sequence of nearly- horizontal, laterally discontinuous, interbedded, unconsolidated, sandy and silty sediments. Horizontal ground water flow occurs almost entirely within the sandy units. The shallow sands behave as an unconfined aquifer, but deeper sands show increasing amounts of delayed yield and confinement, according to KCWA hydrographs.

Because the interbedded silts have some permeability of their own, and because pumping in the deeper zones causes significant downward vertical gradients, the deeper sands obtain a significant fraction of their recharge from the overlying layers. This "leakage recharge" through the permeable silts is augmented by higher- speed, vertical flow at the lateral margins of the silty layers through the more permeable sand facies between layers.

This type of aquifer behavior is complex and difficult to model at the observed scale of variability unless we have much more data than is currently available. However, this aquifer can easily be modeled as a semi-confined (leaky) aquifer with a few simplifications and assumptions. The mathematical theory is available for us to model the project impacts under leaky aquifer conditions, and we consider this to be an acceptable approximation and the best choice among the available alternatives.

Aquifer Model. For this scope of work, there are three mathematical aquifer models from which we are free to choose, i.e., a confined aquifer, an unconfined aquifer, or a leaky aquifer. We must choose one of these three models based on our interpretation of the local geology and hydrology.

The SSS interpretation of the local geology, based on available well logs, consists of complexly- interbedded sandy and silty sediments with reported localized confining layers about 150 ft below ground level and a more extensive confining layer about 550 - 700 ft deep. The cluster- well hydrographs (Figures 6A, 6B, 7A, 7B) which are prepared and presented by the Kern County Water Agency on a monthly basis corroborate the widespread and persistent presence of downward vertical gradients between successively deeper depth intervals which are indicative of leaky aquifers.

We interpret the hydrology and stratigraphy with significant vertical gradients, lateral facies changes, and widespread absence of shallow clay layers to be an aquifer which behaves neither as a confined nor an unconfined aquifer but as a non-ideal leaky aquifer. The combined effects of many thin, laterally discontinuous silty layers contribute to aquifer and well behaviors with delayed yield and both horizontal and vertical flow gradients. Our interpretation differs from the models proposed by Schmidt in 2003 and by the Department of Water Resources (DWR) in 1995, which we summarize for reference in Appendix 1.

Aquifer Parameters. For the leaky aquifer model, we must specify the aquifer dimensions, regional gradient, aquifer storage properties, and aquifer flow properties in both the horizontal and vertical directions. There is a scarcity of reliable parameter data in the Kern Fan area. We have reviewed all of the available data and have found just enough data to make a single estimate of every required parameter. Because of the lack of replicate data, there is an unknown amount of uncertainty in the representativeness of these single parameter values, which is in addition to the uncertainty in the accuracy of these measurements themselves.

We have reviewed the available published sources of parameter values (Appendix 3) and we consider the ID4 well test of December, 2002 to be the best source of a verifiable T & S value for the area of interest. SSS re-analyzed the reported time- drawdown data for one observation well (Appendix 4) and determined the local value of transmissivity to be T = 20,000 ft^2/d and the value of storativity to be S = 0.00056 for the slotted intervals of the tested wells . These values of T & S differ from those published in the original analysis (Schmidt, 2003b) and from other published values in the area. This value of S = 0.00056 from the pump test, disagrees with our calculated value of S which is based on bulk compressibility measurements on sediment samples from the project area of interest. Based on our measurements, we expected a well- test to provide a storativity (S) in the range of $0.003 \leq S \leq 0.015$. Lacking any corroborating data to resolve the discrepancy, we chose to use the measured value of S = 0.00056 from the ID4 December, 2002 well test for this work program.

Based on E-logs, we estimate that the test wells were completed across an estimated 250 ft net sand interval in the local area of the well test, so that the hydraulic conductivity of the sandy strata must be about K_{sd} = 80 ft/d. Based on published RRB data, we have used a value for specific yield of S_y = 0.21 and an average porosity of p = 0.30 for the aquifer sands.

The Hantush leakage factor (B) is a function of the aquifer transmissivity and the vertical flow properties of the aquitard(s) overlying the aquifer. In the project area, the high-permeability zones of the aquifer are sandy sediments and the low-permeability zones are silty sediments. These silty sediments are the aquitards which retard the vertical flow of water between the sandy layers of the aquifer. The vertical flow parameters of interest include the thicknesses and vertical hydraulic conductivities of the silty layers but neither the horizontal nor the vertical hydraulic conductivity can be determined from the local wells or well tests. Based on our measurements and estimates of the relevant properties (Appendix 3), we estimate that the value of B varies in the range of about $1800 \leq B \leq 6000$ and we have used a value of B = 3200 as our base case value.

Both Swartz (1995) and Schmidt (1997) quote generic values for vertical hydraulic conductivity (K_v)for the Kern Water Bank area (see Appendix 1) ranging from .0004 - .0027 ft/d which are within the two orders of magnitude of typical textbook values for silty sediments. Swartz (1995, p.116) indicated that the selected DWR values were guessed at and did not work very well in their computer models and had to be changed to other, unreported values. Schmidt reported (1997, p.7) that their values were determined from long- term well tests performed in the KWB area in 1990 - 1991 but we do not know how this might have been done and Schmidt did not present either the well locations, test methods, test data, or calculations so we cannot independently verify the reported values or their relevance to the ID4 / KT / RRB project area. Except that these reported values fall within the range of expected textbook values for silty sediments, we place no particular credibility in the representativeness of these particular values of K_v. We do not know of any other reported pump test data which provide a determination of the vertical hydraulic conductivity of the local sediments.

There are several reported measured values of vertical hydraulic conductivity Kv_{sand} for both sand and silt samples collected in the area of interest. RRB (Crewdson, 2003) and the City of Bakersfield (COB, 2000) separately reported independent sediment permeability data which are based on laboratory core analyses of shallow unconsolidated sediments which have been retrieved from boreholes down to 120 ft deep. The RRB sand samples had a $Kv_{sand} = 18$ ft/d and the COB sand samples had a $Kv_{sand} = 112$ ft/d. The RRB silt samples had a $Kv_{silt} = 0.038$ ft/d and the COB silt samples had bimodally distributed values of $Kv_{silt} = 0.3$ and $Kv_{silt} = 0.03$ ft/d. Based on these core- sample data, we observe that the local silty sediments are about 500 - 1000 times less permeable than the local sandy sediments.

Based on the Kv/Kh ratio for these sediment analyses and the well-test value of $K_{Hsand} = 80$ ft/d, we estimate that the range of vertical hydraulic conductivity of the silty intervals is about $0.04 < K_{Vsilt} < 0.16$ ft/d with an average estimated value of $K_{Vsilt} = 0.08$ ft/d . Finally, we have estimated the aquitard thickness (b′) based on E-logs and dimensional considerations to be 50 - 100 ft thick and have calculated a range of values of leakance (L′) and Hantush leakage factor (B) accordingly. We have selected an average value of B = 3200 ft for base case drawdown calculations and a range of about $1800 \leq B \leq 6000$ for sensitivity analyses.

For the calculation of drawdown impacts, we have initially assumed that the regional gradient in the test area is zero so that all model impacts are superimposed on an initially flat water table. We set our reference elevation to be zero at the initial water table rather than at ground level or at mean sea level so that all calculated drawdowns are relative to the initial water table. This device allows us to easily observe just the predicted pumping- induced drawdown at any location without the complicating effects of the natural gradient.

However, in order to perform particle trajectory and capture zone analyses, we must superimpose the calculated pumping- induced drawdowns on a realistic approximation of the natural water table gradient. We have based our approximations on observed historical water table behavior.

The Ken Schmidt, 2003a, report presents two different groundwater conditions in their impact analysis of the project area. One condition represents a *northwesterly* water table gradient and a second condition represents a *westerly* water table gradient. Based on our review of KCWA groundwater elevation maps for the area, we have observed an overall change in the groundwater gradient as the climate swings from wet to dry conditions. During a wet cycle, the recharge in the three- mile stretch of the Kern River channel from Allen Rd east to Coffee Rd tends to create a northwesterly component to the overall gradient on the north flank of the river recharge axis such as for the years 1996 - 1998 (Figure 8). During a dry cycle, the absence of recharge in this stretch of river causes a westerly gradient to dominate due to the effects of aquifer dynamics farther to the east such as in years 1991 - 1993 (Figure 9).

This shift may cause contaminant plumes located outside of, but close to, the long term capture zone limit to move into the capture zone. The reverse is not really possible, i.e., contaminant plumes leaving the capture zone, because even though particle trajectories say it is possible, actual contaminant migration invariably leaves in situ residues behind in its pathway which linger as continuing in situ sources of low- grade contamination for many years thereafter. We have included the uncertainty in ground water gradient in our analysis by using a long- term background average ground water gradient behavior within the computational model, based on the observed trends from the KCWA historical ground water elevation maps.

In other model runs we have assumed that the regional water table gradient is 0.002, i.e., about 10 ft per mile either to the west (α = -180 degrees, left azimuth from east), which is typical of dry cycle conditions, or to the northwest (α = -135), which is typical of wet cycle conditions. In these cases, we have set our reference elevation such that the water table approximates the true depth of the water table in the area of interest, which we assumed to be 100 ft deep at the intersection of Stockdale Hwy and Allen Rd for this evaluation.

...

Section V - Drawdown Analysis.

The basic output from a single drawdown analysis is a contour map of the predicted drawdowns in and around the area of the well field. Each map shows the well locations, the contours representing the drawdown for a specified set of pumping parameters, and flowpath particle trajectories for a specified duration of pumping. The maps cover an area of approximately ten square miles centered on the middle of the well field. Using local (east, north) coordinates in units of feet, the local origin (0,0) is at the intersection of Stockdale Hwy and Allen Rd, the southwest map corner is located at (-8400, -4600), and the northeast corner (+11600, +10600).

The predicted drawdowns from this work program are significantly different than the predicted drawdowns from three other recent impact analyses by other workers in five respects. First, SSS modeled the aquifer as a leaky aquifer rather than as a confined aquifer. Second, SSS used the superposition method versus the so-called centroid method used in the other studies. Third, SSS's parameter values are different than those of the other studies, and incidentally are different in such a way as to increase the calculated drawdowns, all else being equal. Fourth, the leaky aquifer model which SSS used predicts that the water levels will decline and then stabilize at a static, steady- state drawdown at least for a while, compared to the other forecasts which predict that water levels will continue to decline as long as pumping is continued. Fifth, for SSS's choices of aquifer model and aquifer parameters, the predicted drawdowns are significantly less than the predicted drawdowns from these other studies.

<u>Expected results.</u> We expect at any moment after pumping has begun that a cone of depression will form around each well and that the cone of depression will deepen and expand outward with time, subject to certain limits. We expect at any moment, that the drawdowns will be larger close to the wells and smaller farther away from the wells. We expect at any location that drawdown increases as the duration of pumping increases. We also expect for any specified time and location, that the drawdown will be larger for higher pumping rates and

smaller for lower pumping rates. We also expect that for any location that is within the radii of influence of more than one pumping well, that the observed drawdown will be the sum of the individual drawdowns caused by every pumping well superimposed at that location.

What is not as intuitive is the expected drawdown behavior depending on the choice of aquifer model. If the aquifer is fully confined or fully unconfined, the drawdowns will continue to decline indefinitely. If the aquifer is semi-confined with leakage recharge from the overlying layers, the observed qualitative behavior will be more complicated. For a short period of time, the aquifer will behave as a confined aquifer, meaning that the observed drawdowns near each of the wells will decline quickly and with the same time - distance relationship as is predicted for a confined aquifer with the same values of T & S. Thereafter, the water table will decline at a decreasingly slower rate than predicted by the confined-aquifer model until the water table stops falling altogether.

After an undetermined time period of leaky behavior during which there is little or no observed drawdown despite continued pumping, we expect that the water table will once again start to decline at a rate which is consistent with the de-watering of an unconfined aquifer with the assumed values of T & S_y. The durations of each of these behavioral phases may be estimated but the calculated times of transition are not particularly precise because of the inability to predict future recharge. This project can be in leaky steady state for a very long time if the shallow aquifer is consistently recharged. Once this program has begun, a properly designed well- testing and monitoring program will provide a wealth of new understanding of the aquifer, well beyond what we are able to model with the small parameter set which is available at this time.

ID4 observed decline- and- stabilize behavior in the December, 2002 well test after only 6 days of pumping, which they attributed to the onset of ground water recharge in the nearby Kern River channel. Perhaps. But the observed stabilization of the water table more likely represents the leaky aquifer condition in which all pumped water was being supplied by downward leakage from the overlying layer and the well pumpage had achieved balance with the rate of vertical recharge.

For the ID4 December, 2002, well test, the drawdowns reportedly stopped declining in some wells after only six days of pumping, which was attributed to ongoing recharge in the

Kern River channel. But the math doesn't really support this logical sounding explanation. The combined recovery rate of the six pumping wells was about 95 af/d but the river was recharging only about 20 af/d. The river recharge could only have supplied about 20% of the total recovery rate even if there had been no delay whatsoever in moving recharge water from the river to the producing intervals of these six wells. If the river had really been supplying the pumped water, the river would have dried up! Most of the recharge that caused the drawdowns to stop declining must have come from a "real- time" source other than the Kern River channel.

In our opinion, the available data suggest that the more likely explanation of the drawdown behavior which was observed during the ID4 December, 2002 well test is that the aquifer was behaving like a leaky aquifer and the drawdowns stabilized as soon as the zone of depression had spread sufficiently to capture enough recharge leakage to balance the recovery rate of the test wells. In this model, the river recharge is essential to keep the shallow water table aquifer replenished over the long term against the loss from this zone due to leakage, but based on theoretical and practical grounds we would not say that the river was the direct cause of the observed stabilization of drawdowns during the December, 2002, well test. However, subject to confirmation during future project operation, this previously- observed behavior suggests that the leaky aquifer model is an appropriate approximation to the aquifer under the project area.

<u>Computed results.</u> The base case operating scenario is to pump all seven wells at a combined flow rate of 90 af/d for 300 days to recover 27,000 af from aquifer storage. The base case aquifer model is a leaky aquifer with T = 20,000 ft^2/d, S = 0.00056, $\phi = 0.30$, and B = 3200 ft. The base case water table model is a horizontal water table for impact analysis, and two cases of sloping water table; a planar gradient of pointing either in the direction N45°W or N90°W with or without river recharge, for zone of capture and particle trajectory analyses. We present the calculated drawdowns for the base case and for other cases representing a broad range of parameter values in the figures and appendices. The calculated drawdown at any specific location for any specified set of conditions may be read directly off the respective drawdown contour map.

<u>Base Case.</u> If the local aquifer is a *leaky aquifer* as we have modeled it under the base case conditions, then after 300 days of pumping, the induced steady state drawdowns at

distances of 1000, 3000, and 5000 ft from the well field perimeter are predicted to be 16 - 24 ft, 8 - 12 ft, and 3 - 5 ft, respectively (Figures 10 & 14). The model predicts that the entire drawdown will take place in the first 10 - 20 days and then remain steady or nearly so thereafter. These maximum, steady-state drawdowns will remain unchanged even for multiple consecutive years of well operation as long as the shallow zones continue to be recharged and are not dewatered. The actual operating drawdowns will be similar to these calculated drawdowns if our selected parameter values are representative of the aquifer when the project actually begins.

This steady-state drawdown condition does not come for free, i.e., the water must come from somewhere. The real- time recharge to the aquifer comes at the expense of the shallow zone which will experience a decline in water table of 30 ft near the well field for the duration of pumping or more if this layer is not recharged.

There are five domestic water supply wells and four municipal water supply wells within approximately 1,000 - 1,200 ft of any one of the project wells (Figure A2). There are another nine wells 1,200 - 3,000 ft away, and another 9+ wells within 3,000 - 5,000 ft. These wells include six City of Bakersfield wells, at least four other municipal water supply wells, and five non-project ID4 wells within this zone of influence. None of these other entities except ID4 has performed or published a well placement analysis or impact analysis on their wells, to our knowledge.

The potential impacts of these nearby wells on this ID4/KT/RRB project is currently outside our scope of work, but the bigger wells will certainly have drawdown impacts and particle trajectory - capture zone impacts in the project area which we have not incorporated into this study.

The ASR project is designed to put a wet- year water supply into the ground and then recover it in some future dry year, so there is little likelihood of recharge and recovery happening simultaneously. However, as long as the project puts as much water in the ground as they take out, the net basin impacts of recovery will be exactly compensated for by the mounding impacts of recharge, so there will be no net long term effect on the basin no matter how far apart recharge and recovery are separated in time. The purpose of this study is to determine the magnitude of the potential drawdowns over a single season of pumping, and whether or not these short- term temporary impacts are large enough to exceed project impact criteria.

<u>Base Case Specific Capacity of the Pumped Wells.</u> The specific capacity (SC) is defined as the ratio of discharge rate to drawdown within a pumping well and is used by local engineers as a measure of well performance from which other parameters are calculated. Unfortunately SC is not a constant and varies with pumping time, length of completion interval, hole diameter, and well efficiency, so it is not an effective measure of anything without making the corrections for each of these factors. We can calculate the theoretical specific capacity (SC) of the project wells for the steady- state leaky aquifer condition from the selected base case parameters for purposes of preliminary pump parameter selection. Normally for pump design purposes, we would recommend using actual drawdown data from nearby pumping wells as the best predictor of well performance, but we can calculate a value as well.

For an aquifer transmissivity of 20,000 ft/d which is based on a re-calculation of the ID4 December, 2002, well test and a Hantush leakage factor of $2600 \le B \le 3200$, we estimate the expected *steady- state* project- well specific capacity to be around $0.15 \le SC \le 0.25$ cfs/ft. For all pumping times less than required to reach steady- state, the observed SC will appear to be larger, perhaps much larger than this predicted final value. For example, the reported time-drawdown data from a 12-hour ID4 pump test of nearby ID4 well #12 (29s/26e- 36Q02) gives uncorrected SC values in a range equivalent to $0.46 \le SC \le 0.60$ cfs/ft. Previous workers have used such non-stabilized values of SC to report calculated, but undocumented, values of aquifer transmissivity as high as 40,000 - 60,000+ ft^2/d in this immediate area. Until more documented and verifiable data are available in the project area, we will remain skeptical of these values.

<u>Modified Base Case.</u> Because of the uncertainties in the actual aquifer conditions, the actual operating drawdowns may be different than the calculated base case values. We have already acknowledged that there is considerable uncertainty in the few data available to us. Since the accuracy of the impact calculations depends primarily on the values of T, S, and B, we have varied the base case parameters within the credible ranges of possible values and have re-calculated the drawdowns for these other parameter values.

In general terms, we have selected base case values of T, S, and B based on actual measurements but which are at the lower end of their respective ranges of possible values. If the true aquifer transmissivity (T) is higher than our value, then the drawdowns will be less than predicted, but the zone of impact will extend farther out than predicted. If the true aquifer storativity (S) is higher than our value, then the drawdowns will propagate outward more slowly but the final drawdowns will not change in the long run. If the actual leakage factor (B) is higher than our value, then the actual leakage from the shallow zone into the underlying aquifer will be less than we've assumed, the aquifer will behave more like a confined aquifer than expected, and the drawdowns will be larger than predicted. These qualitative trends can be evaluated by modifying the base case parameters through a range of values and observing the changes in calculated drawdowns.

We have assembled a catalog of calculated drawdown maps (Appendix 6) for modified base case parameters and have summarized some of the interesting results in Appendix 6.

Limiting Cases. In general terms, the limiting cases for any impact analysis occur when pumping continues for a very long time, i.e., approaching the condition of steady - state. If the aquifer extends uniformly outward for a very long distance away from the area of operations (i.e. what we call infinite extent) and the aquifer is either confined or unconfined, then the drawdown will never actually achieve steady state but will, instead, continue declining forever at a very slow rate. This rarely occurs in real life because either pumping stops, or the drawdown extends outward until a recharge boundary is reached and then the drawdown stabilizes, or pumping wells nearly drain the aquifer dry and the theory falls apart. Other things which can happen along the way are outside the scope of this discussion.

In these two hypothetical steady state cases, drawdown is greater and propagates farther in the confined aquifer than in the unconfined aquifer. In the ID4 area of interest, the aquifer behavior in either case would achieve about 90% of steady state behavior by the time the wells have been pumped for 300 days. One implication of this is that once the aquifer behavior has achieved steady state, no further (significant) drawdown will occur even if pumping is continued for a long period of time. And this is true of the ID4 project. The calculated drawdown due to three consecutive years of base case pumping is only a few feet deeper than the drawdowns at the end of the first year.

For comparison purposes, the hypothetical steady state drawdowns for fully confined and fully unconfined conditions are as follows. If the local aquifer were *fully confined*, then after 300 days of pumping the drawdowns within the aquifer would be greater than 100 ft at a distance of 1000 ft from the wells and 74 - 82 ft at a distance of 5000 ft from the wells (Figure 11). If the local aquifer were *unconfined*, then after 300 days of pumping the drawdowns within the aquifer would be as much as 60 - 70 ft at a distance of 1000 ft from the wells and 44 - 50 ft at a distance of 5000 ft from the wells (Figure 12).

Section VI - Flow Trajectory and Capture Analysis.

<u>Particle trajectories.</u> A particle trajectory represents the hypothetical flowpath of a water molecule under ideal flow behavior, i.e., ignoring the effects of dispersion, flowpath tortuosity, heterogeneity, etc. We can calculate particle trajectories in downgradient or upgradient directions, which we refer to as forward or reverse particle tracking, respectively. In our computational models we assume that the aquifer is horizontally isotropic so that particle trajectories are always perpendicular to water level contours. For this project we used reverse particle trajectories to determine the shapes and extents of the capture zones for each of the pumping wells in the well field for different pumping durations.

The flowpath behavior in the project area under pumping conditions represents non-Darcy flow in which the particle trajectories of adjacent water molecules are not parallel and do not travel at constant velocity. We can easily calculate the water flow velocity at any point at any time but it is not particularly useful to do so. The best generalization we can make is that groundwater flows the slowest at the perimeter of the capture zone, the fastest as it approaches a well, and at intermediate velocities in between because it varies with the gradient which varies with the distance and position relative to the well field.

It is much more useful to be able to map the flowpaths that aquifer water takes as it flows toward a well because any potential constituents of concern in the groundwater will follow the same flowpaths. One important use of particle trajectory mapping is for designing contaminant- detection monitoring programs so that the operator can place the monitoring wells in the likely flowpaths from known or suspected contaminant sources.

Capture zones. A capture zone is the enclosing perimeter of the actual bulk volume of the aquifer from which a pumping well extracts water over a specified time period. For a confined or semi- confined aquifer, the capture zone is a vertical cylinder centered on the well and bounded by the confining layers at the top and bottom of the aquifer. The radius of the capture zone increases as the pumping rate and/or duration increase. The shape of the capture zone will be distorted by the presence of other wells and/or recharge boundaries but it will always have a fully enclosing perimeter. The method of reverse particle tracking will always provide a means to map the shape and extent of the capture zone for a specified pumping duration.

The capture zone (CZ) is the cylindrical volume of the aquifer centered on the well field from which ground water is actually removed by pumping over a specified time. The CZ is not the same volume or the same shape as the cone of depression which merely shows the distribution of head within the aquifer. The importance of mapping the capture zone is for purposes of evaluating water quality, particularly the potential for contaminant capture. We have mapped (Figure 13) and tabulated (Figure 15) the approximate locations of the expanding capture zone for continuous pumping for pumping times from 1 - 30 years for an aquifer with a northwesterly water table gradient (Figure A9).

A slug or plume of contamination which is inside the capture zone will arrive at the well field within the specified pumping time if the contaminant moves at the same speed as the groundwater. For many contaminant constituents, this assumption is false, since the processes of dispersion, retardation, and attenuation affect the flow velocity of contaminants in ground water. There are no rules of thumb in this regard without specifying the contaminant of concern, but the capture zones which are based on the flow velocity of the ground water form the base case of any contaminant capture analysis. Sierra Scientific Services has performed contaminant transport modeling for other clients, but it is outside this scope of work, particularly since no zones of contamination have been reported in the immediate project area.

Based on the base case operating scenario, the following table gives the approximate dimensions of the well field capture zone for various hypothetical periods of continuous pumping. The distances are measured from the nearest edge of the well field and not the center.

Capture Zone Perimeter Distances, measured from well field perimeter.

Pumping Time (yr)	Downgradient Distance (ft)	Upgradient Distance (ft)	Rate of outward Expansion (ft/yr)
1	1000	1400	1400
2	1500	2100	700
5	2300	3500	560
10	3000	5200	340
20	3500	5800	80
30	3500		
∞	~3500		

Based on these data, the well field will never capture water (or contamination) from plumes or sources which are farther downgradient (to the northwest) than about 3500 ft regardless of the duration of pumping. In real life, a capture zone only continues to expand upgradient until it reaches a recharge boundary. For this well field, the upgradient is actually quite close to the south and southeast because the Kern River channel is only about 3,000 - 6,000 ft upgradient. Under the natural gradient alone, with a homogeneous groundwater flow velocity of about 0.5 ft/d, it would take a water molecule of river recharge about 30 - 40 yr to reach the wells and under continuous pumping it would still take 10 - 20 yr. If there are no sources of contamination exist between the well field and the Kern River channel, then the ground water arriving from this direction will always have the same general water quality as Kern River water.

5. References

local work products

City of Bakersfield, April, 2000, Rosedale Area Recharge, Review of Historic Records & Percolation Test Data, State Route "58" Alignment, Kern River Freeway, Final Report, prepared by the City of Bakersfield Water Resources Dept.

Crewdson, Robert, A., 20 January, 2003, Determination of Aquifer Storage Capacity for the Rosedale -Rio Bravo Water Storage District, Bakersfield, California, Sierra Scientific Services, Bakersfield, CA.

Crewdson, Robert, A., 20 March, 2003, A Review of the ID4 Kern Parkway Recovery Well Impact Report of January, 2003, Bakersfield, California, Sierra Scientific Services, Bakersfield, CA.

Crewdson, Robert, A., 20 November, 2003, An Evaluation of the Long Term Water Supply of the Rosedale -Rio Bravo Water Storage District, Bakersfield, California, Sierra Scientific Services, Bakersfield, CA.

Schmidt, Kenneth D., November 27, 1997, Hydrogeologic Conditions for Development of the Maximum Recovery Plan for the Kern Water Bank Authority, revised report, Kenneth D. Schmidt & Associates, Fresno, CA.

Schmidt, Kenneth D., September 22, 1998, Maximum Recovery Plan for the Pioneer Kern Fan Project, draft report, Kenneth D. Schmidt & Associates, Fresno, CA.

Schmidt, Kenneth D., January, 2003, Groundwater Conditions and Potential Impacts of Pumping for the ID-4 Kern Parkway and Rosedale - Rio Bravo WSD Projects, Kenneth D. Schmidt & Associates, Fresno, CA.

Schmidt, Kenneth D., February 28, 2003, Supplement to the Groundwater Conditions and Potential Impacts of Pumping for the ID-4 Kern Parkway and Rosedale - Rio Bravo WSD Projects, aka Allen Road Well Field December 2002 Pump Test, Kenneth D. Schmidt & Associates, Fresno, CA.

Swartz, Robert, 1995, Development and Calibration of the Kern Fan Ground Water Model, DWR San Joaquin District Report, July, 1995.

technical references

Bird, R. Byron , Stewart, W. E., and Lightfoot, E. N., 1960, Transport Phenomena, John Wiley

Fetter, Charles W., 1994, Applied Hydrogeology, Prentice Hall.

Domenico, Patrick A., and Schwartz, Franklin W., 1990, Physical and Chemical Hydrogeology, John Wiley & Sons.

Bouwer, Herman, 1978, Groundwater Hydrology, McGraw - Hill.

6. General Disclaimer

Sierra Scientific Services (SSS) provides a standard work product which includes the performance of work, a copy of the raw data and processed data, and a descriptive report. SSS represents that the performance of work, and the collection, reduction, and analysis of data are by methods which are consistent with standard scientific practice. We represent that the data are accurate in the sense that they can be verifiably duplicated within the limits of uncertainty by a competent investigator using the same methods under the same conditions. SSS does not represent that the data are necessary or sufficient to provide any characterizations beyond that which is consistent with the type, amount, and distribution of data. SSS's sole liability with regard to the standard work product is limited to the correction of errors in our standard work product within the original scope of work. In no event shall SSS be liable for data from other sources even if, for the convenience of the Client, we choose to report it for any reason within our work products. Our sources for the definition of standard scientific practice are available upon request.

Sierra Scientific Services (SSS) provides an extended work product which includes ideas, concepts, theories, hypotheses, speculations, interpolations, extrapolations, interpretations, conclusions and recommendations, collectively referred to as opinions. SSS uses methods of logic and inference which are consistent with standard scientific practice to generate the extended work product. It is SSS's opinion that we provide the Client with a better understanding of the data with our opinions than without them. SSS represents that these opinions are consistent with the overall body of data in the standard work product and are consistent with the workings of nature as we know it. SSS does not represent that these opinions are the only opinions that may be inferred from the data. SSS does not represent that these opinions reflect the true nature of the site. There is no warranty, express or implied, that these data and opinions are useful or usable for a specific purpose.

This work was done at the request of the Client. SSS does not represent or warrant that the data and/or opinions are necessary or sufficient to characterize the site, or that the data or opinions represent the true nature of the site apart from the accuracy of the actual observations. SSS advises that the Client can obtain higher confidence and better characterization through additional work. SSS advises that until the project is fully characterized with high confidence, the Client may be at risk of making inefficient, non-optimum, uncertain, or incorrect decisions based on existing data and opinions.

These representations set forth here are in lieu of all other representations or warranties, whether express or implied, with respect to the standard and extended work products of this report. In no event shall SSS be liable for any special, indirect, incidental, or consequential damages, related to using or not using these work products.

Figures

1. Project Location Map.
2. Project Well Location Map.
3. Well Placement Arrays.
4. Recovery Well Operating Scenarios.
5. Impact Analysis Parameters.
6. KCWA Hydrograph for cluster well 29/26-35H (RRB short & long term).
7. KCWA Hydrograph for cluster well 30/26-04J (Pioneer short & long term).
8. KCWA 1998 Groundwater Elevation Map.
9. KCWA 1993 Groundwater Elevation Map.
10. Base Case Drawdown Map, leaky aquifer at t = 300d.
11. Base Case Drawdown Map, confined aquifer at t = 300d.
12. Base Case Drawdown Map, unconfined aquifer at t = 300d.
13. Capture Zone Map, leaky aquifer with GW gradient for t = 1, 2, 5, 10, 20, 30 yr.
14. Table of Drawdowns versus Distance.
15. Table of Capture Zone Perimeter Distance versus Time.

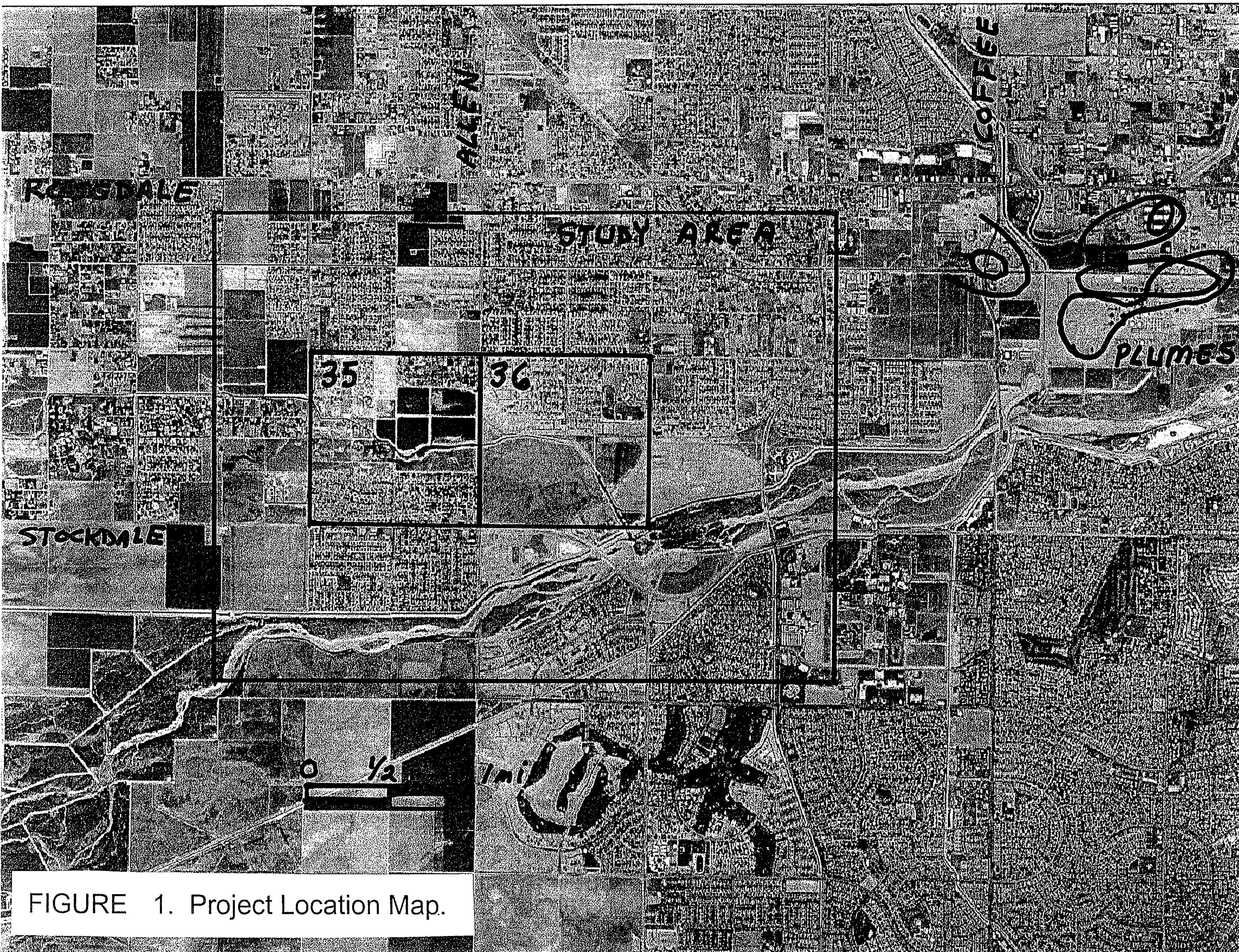


FIGURE 1. Project Location Map.

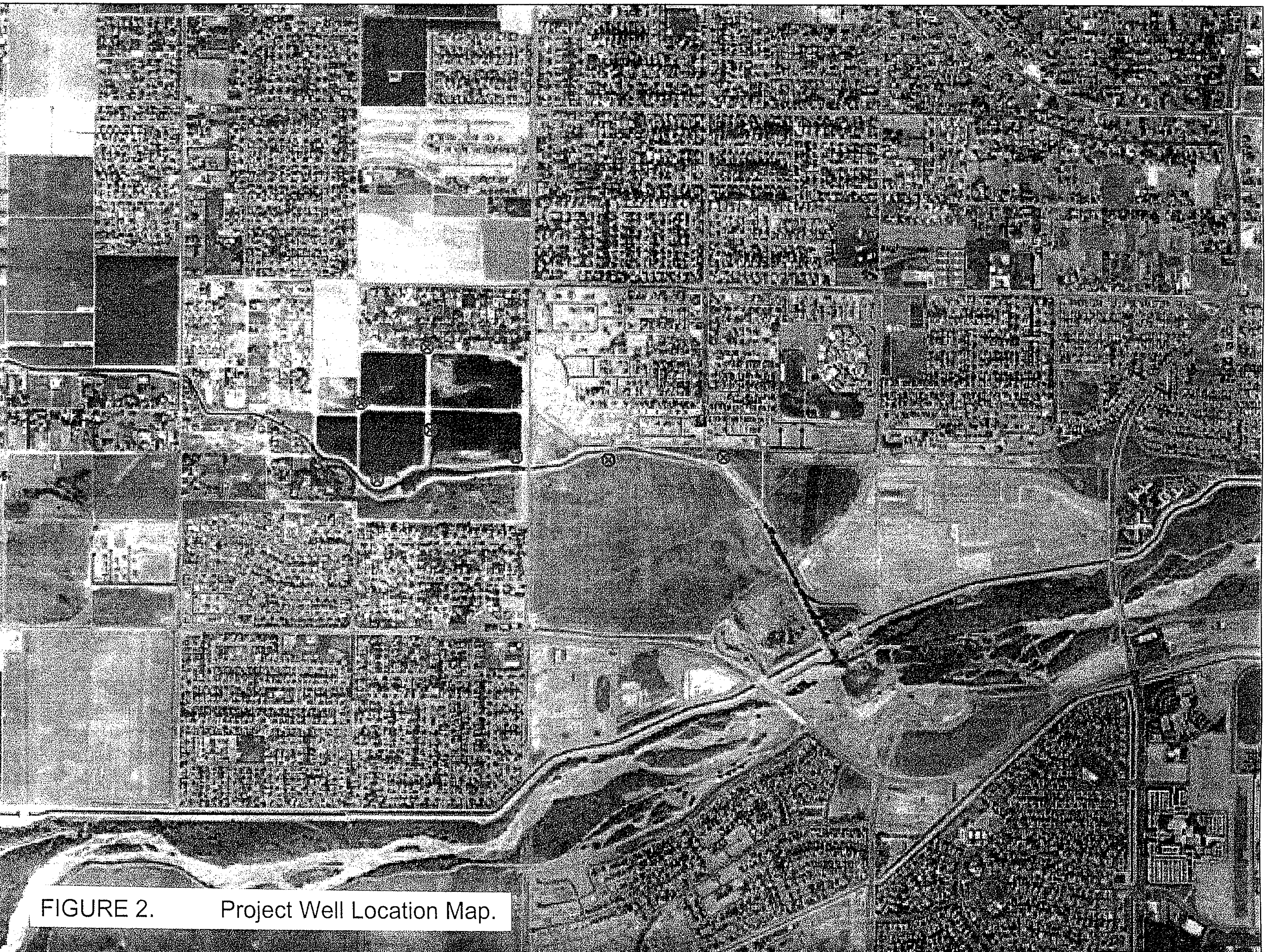


FIGURE 2. Project Well Location Map.

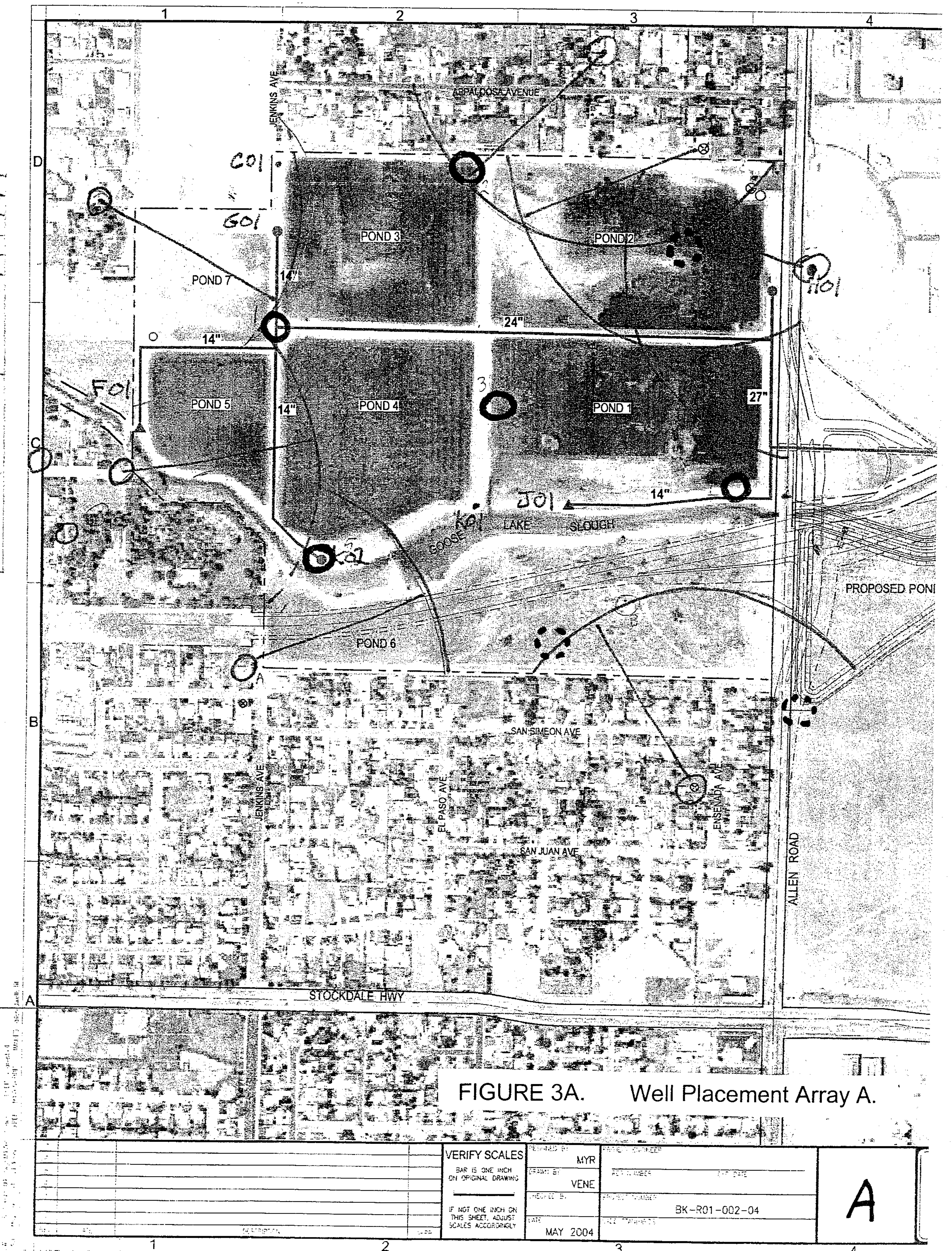


FIGURE 3A. Well Placement Array A.

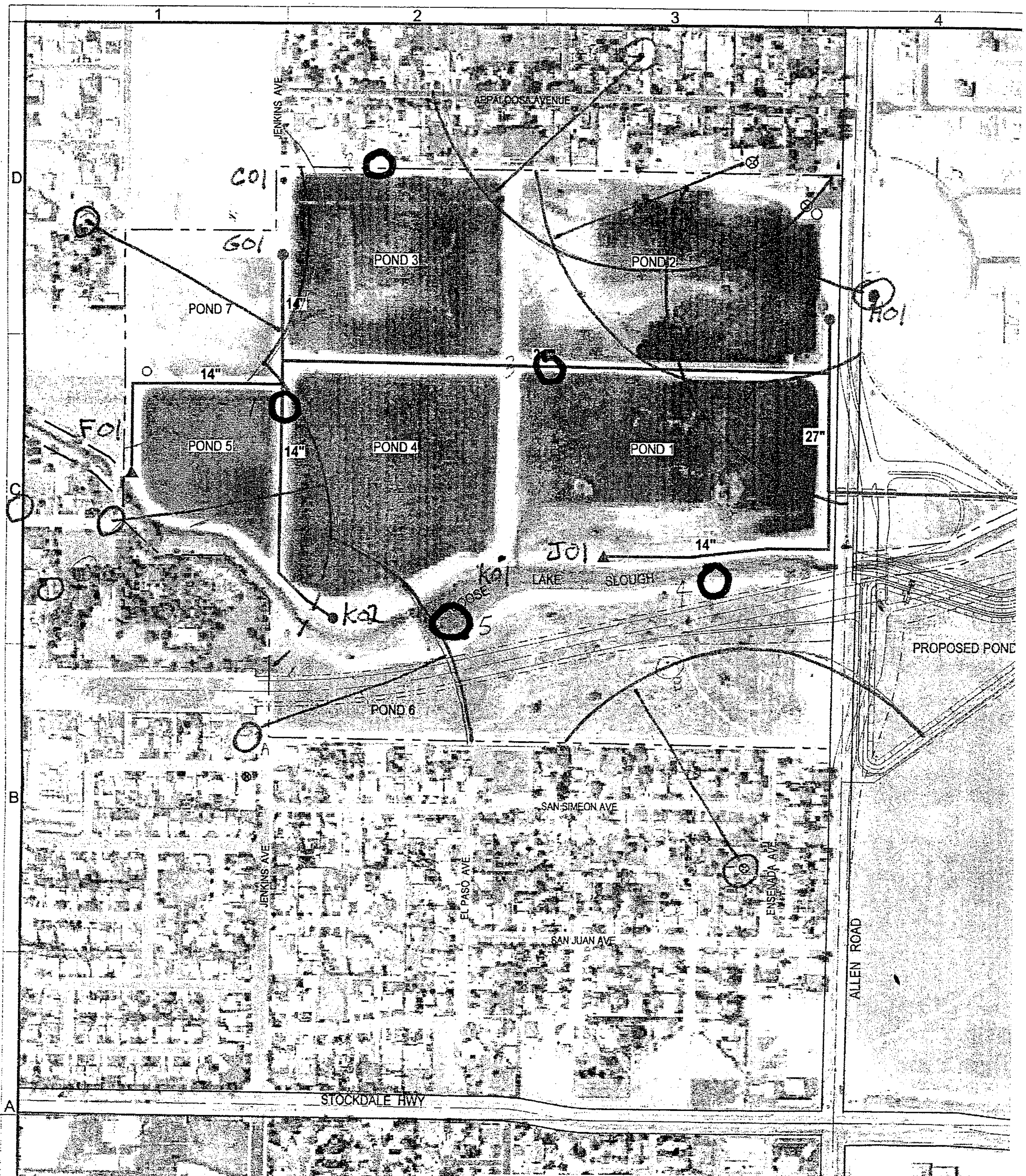


FIGURE 3B. Well Placement Array B.

VERIFY SCALES BAR IS ONE INCH ON ORIGINAL DRAWING IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY	MYR VENE MAY 2004	BK-R01-002-04	B

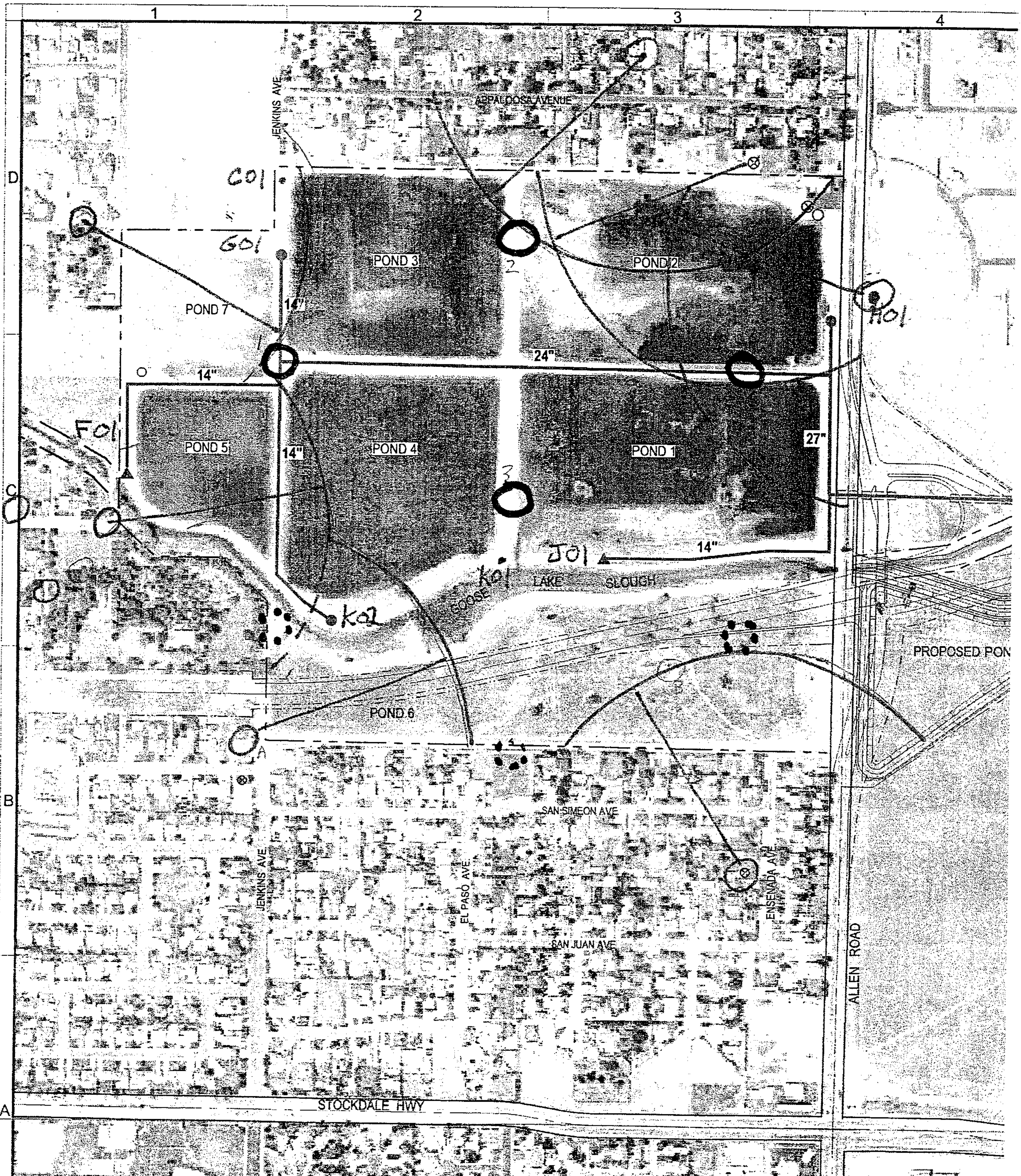


FIGURE 3C. Well Placement Array C.

VERIFY SCALES

BAR IS ONE INCH ON ORIGINAL DRAWING

IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY

DESIGNED BY MYR

DRAWN BY VENE

CHECKED BY

DATE MAY 2004

PROJECT ENGINEER

PROJECT NUMBER BK-R01-002-04

C

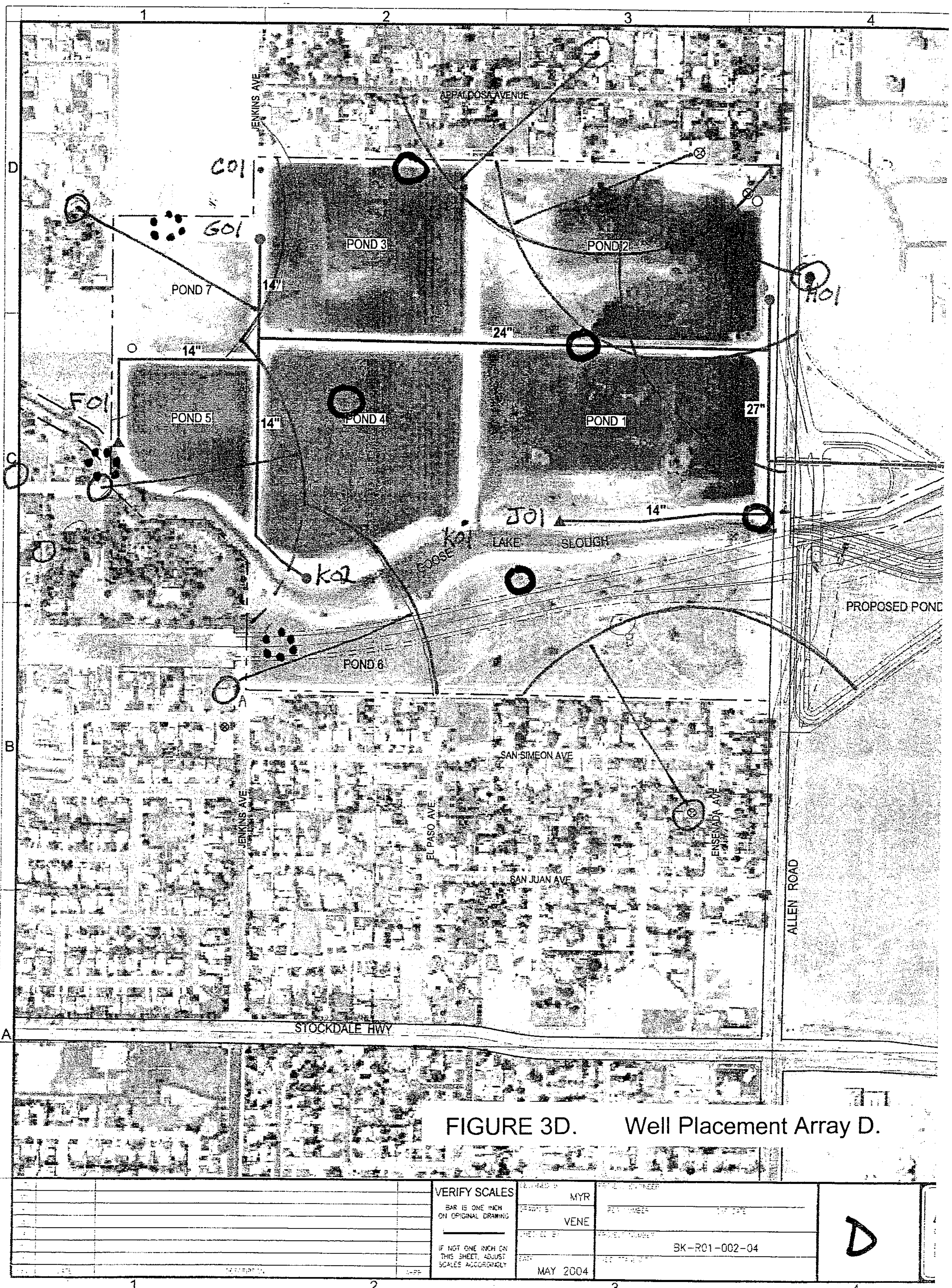


FIGURE 3D. Well Placement Array D.

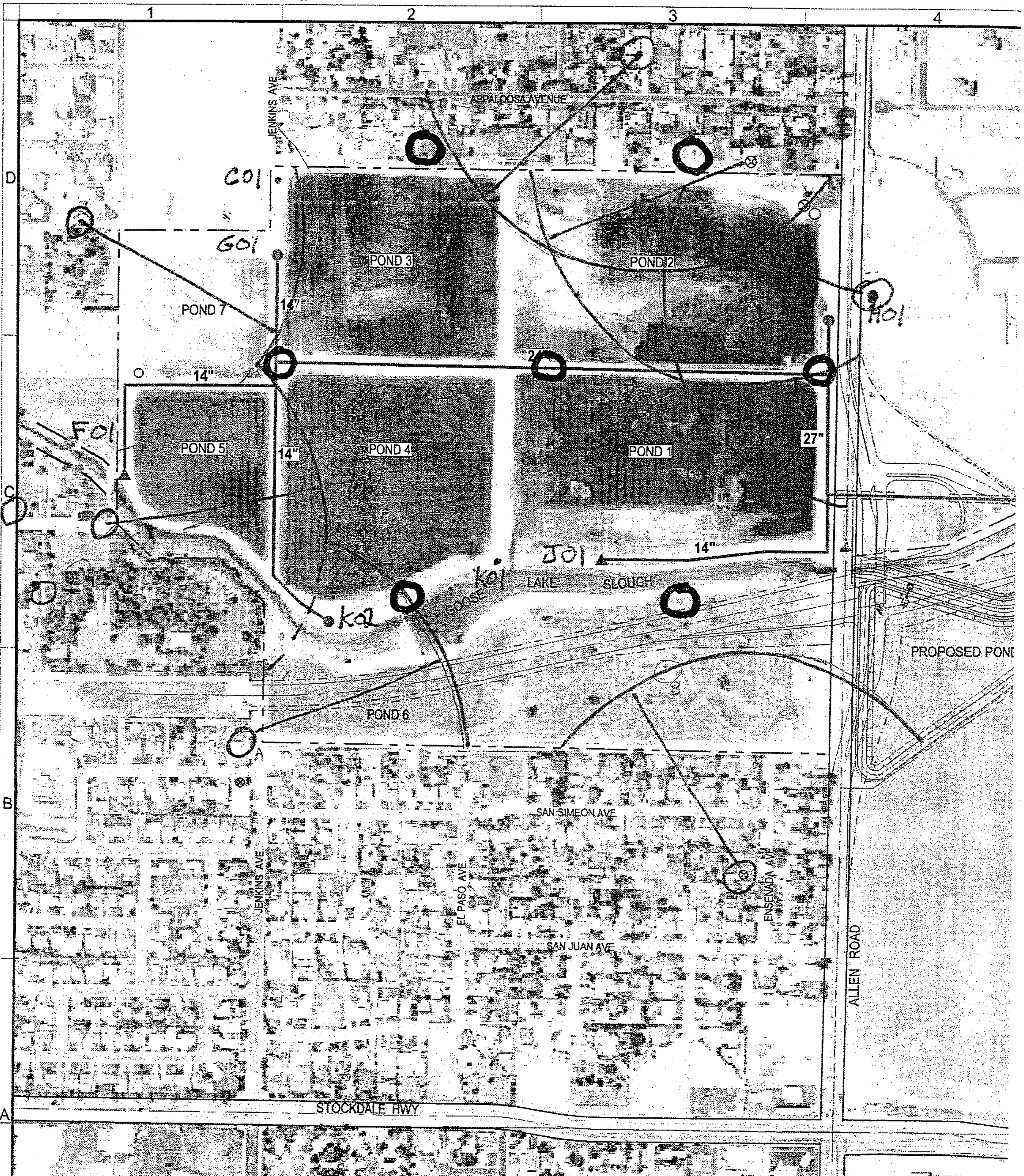


FIGURE 3E. Well Placement Array E.

VERIFY SCALES BAR IS ONE INCH ON ORIGINAL DRAWING IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY	MYR VENE MAY 2004	BK-R01-002-04
		E

ID4 / KT / RRB Project Operating Scenarios.

	Wells	Rate (af/d)	Duration (yr)	Duration (d/yr)	Recovery (af/yr)	Recovery (% full)
base case (100%)	7	90	1	300	27000	100%
3-yr base case	7	90	3	300	27000	100%
Half base case 1 (t = 1/2)	7	90	1	150	13500	50%
Half base case 3 (Q = 1/2)	7	45	1	300	13500	50%
Half base case 3 (RRB wells)	5	50	1	300	15000	56%
Half base case 4 (ID4 wells)	2	40	1	300	12000	44%
Quarter base case 2 (t = 1/4)	7	90	1	75	6750	25%
Quarter base case 3 (Q = 1/4)	2	22.5	1	300	6750	25%
Quarter base case 1 (1 well each)	2	30	1	300	9000	33%

FIGURE 4. Recovery Well Operating Scenarios.

ID4 / KT / RRB Impact Analysis Parameters.

Property	Sym.	Value	Units
Aquifer Hy. Conductivity (Hor)	K(h)	80	ft/d
Aquifer Hy. Conductivity (Vert)	K(v)	8	ft/d
Aquifer Thickness	H	250	ft
Aquifer Transmissivity	T	20000	ft^2/d
Aquifer Specific Yield	Sy	0.21	v/v
Aquifer Specific Storage	Ss	4.1E-05	ft^-1
Aquifer Storativity	S	0.00056	v/v
Aquifer Porosity	phi	0.3	v/v
Aquitard Hy. Conductivity (Vert)	Kv'	0.08	ft/d
Aquitard Thickness	H'	40	ft
Aquitard Leakance	L'	0.002	d^-1
Hamtush Factor	B	3200	ft
GW gradient 1	G	0.002	@ ra = 270
GW gradient 2	G	0.002	@ ra = 315
GW gradient 2 w/rvr rchg	G	0.001	@ ra = 300
river recharge	qrvr	210	ft^2/d
pond recharge	qpond	1	ft/d

FIGURE 5. Impact Analysis Parameters.

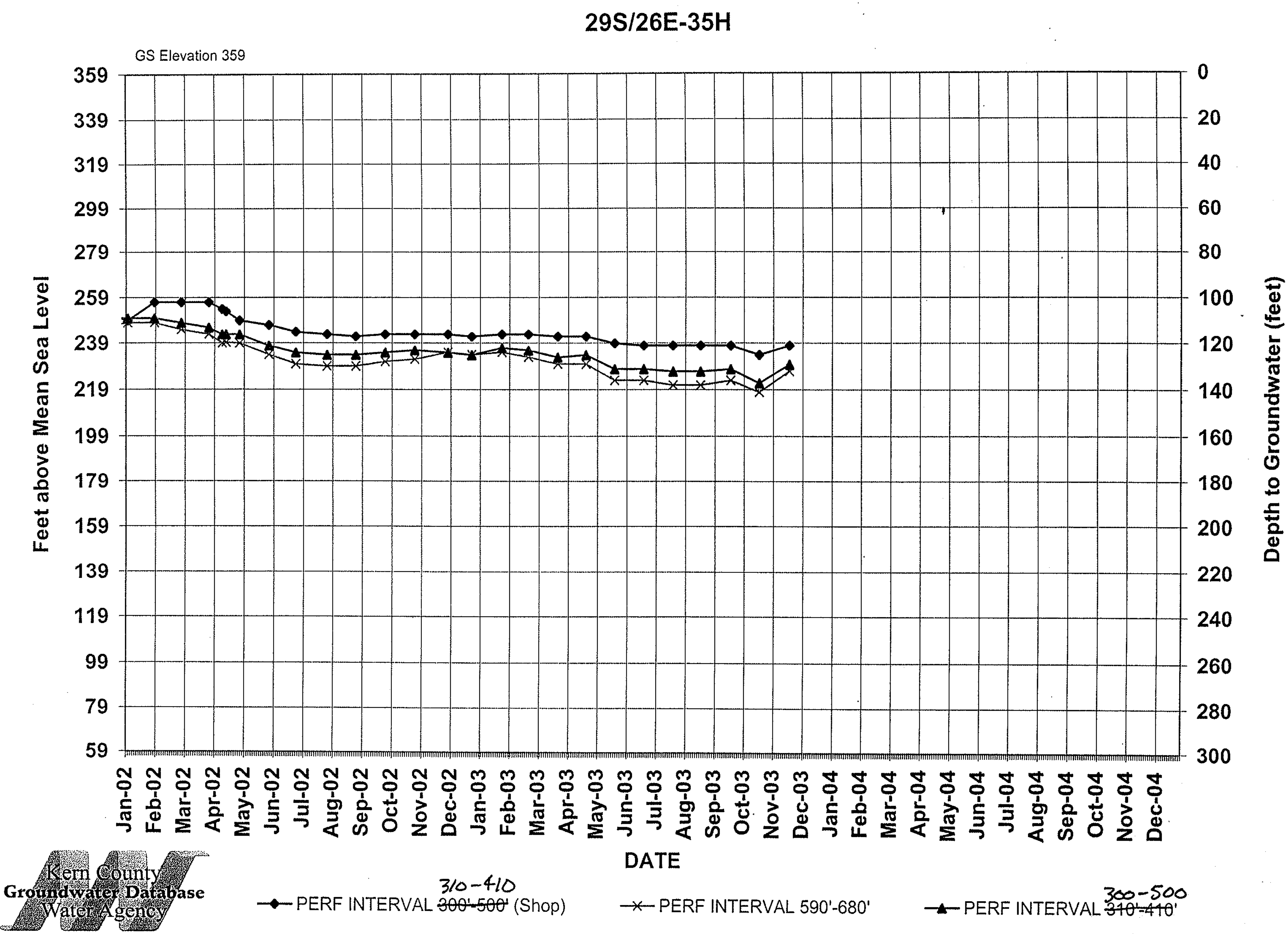
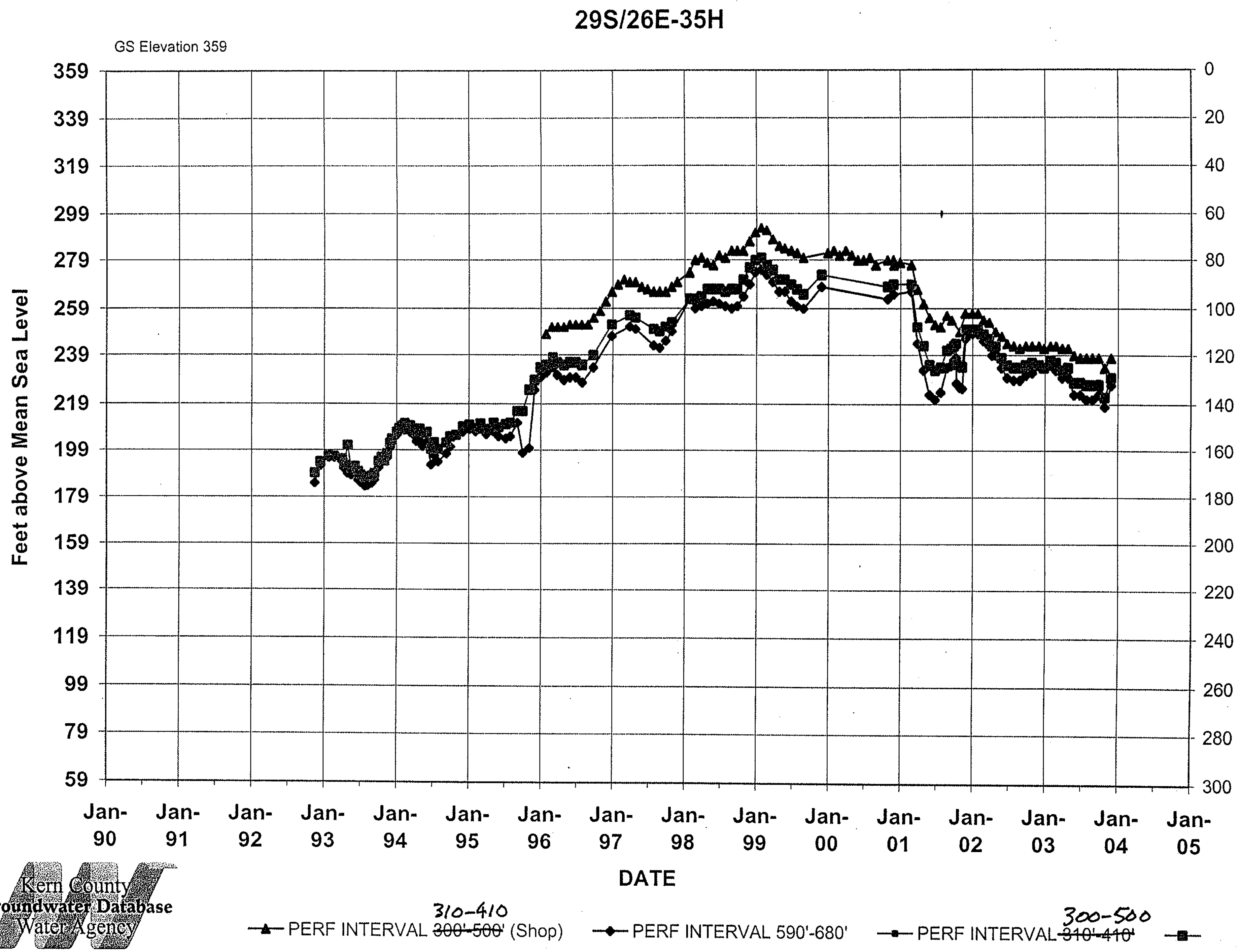


FIGURE 6a. KCWA Hydrograph for cluster well 29/26-35H (RRB short term).



Kern County Groundwater Database Water Agency

FIGURE 6b. KCWA Hydrograph for cluster well 29/26-35H (RRB long term).

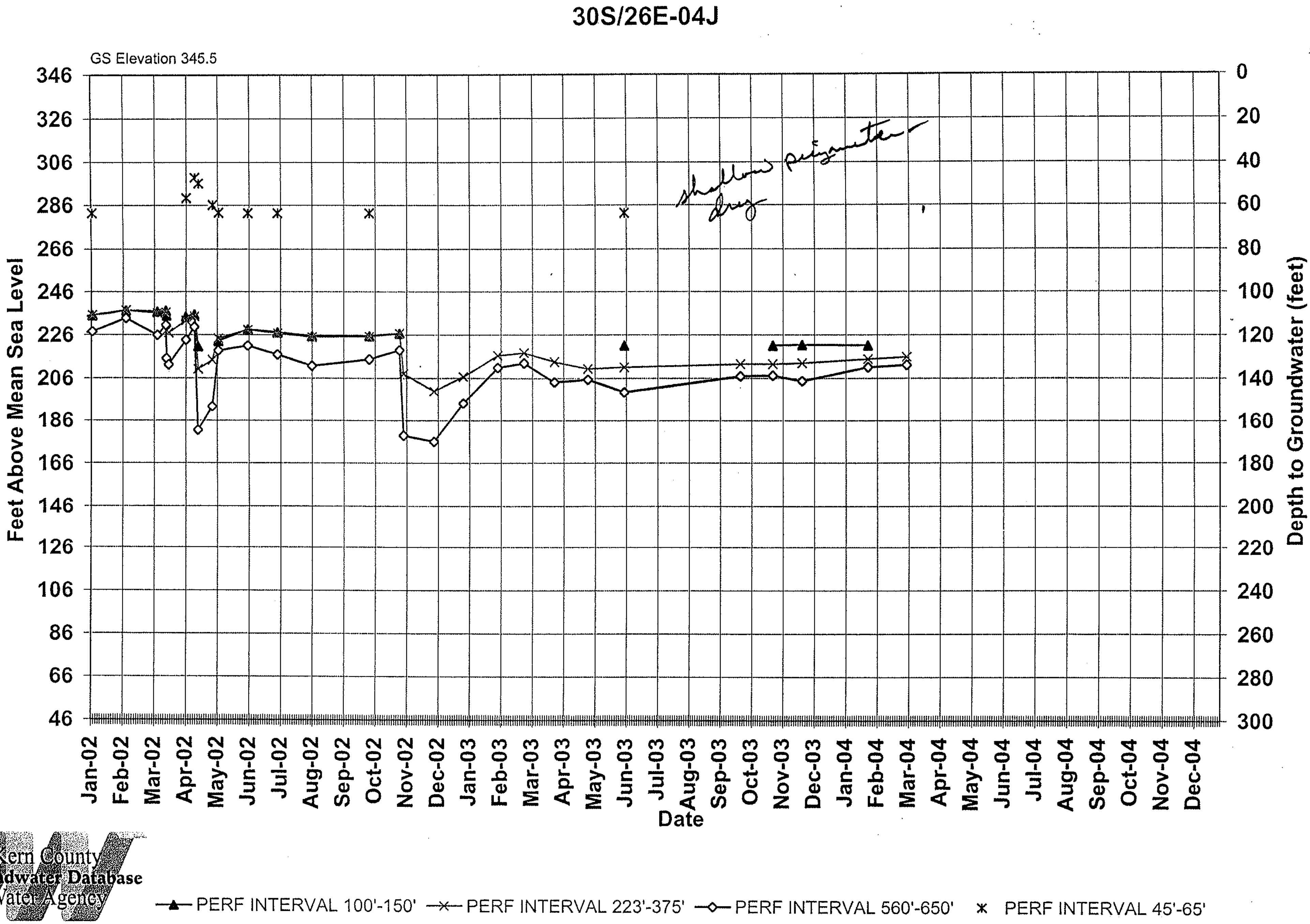


FIGURE 7a. KCWA Hydrograph for cluster well 30/26-04J (Pioneer short term).

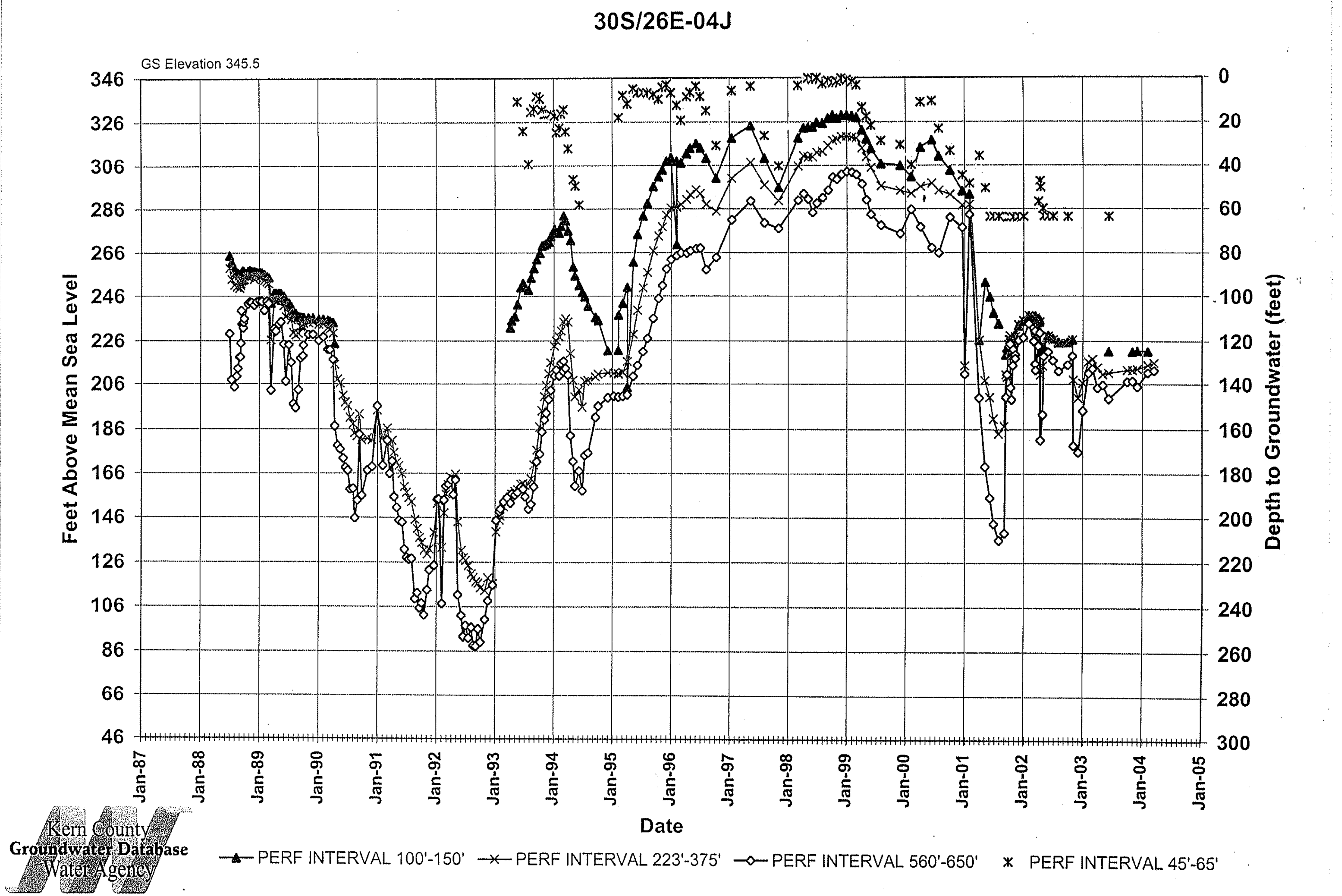


FIGURE 7b. KCWA Hydrograph for cluster well 30/26-04J (Pioneer long term).

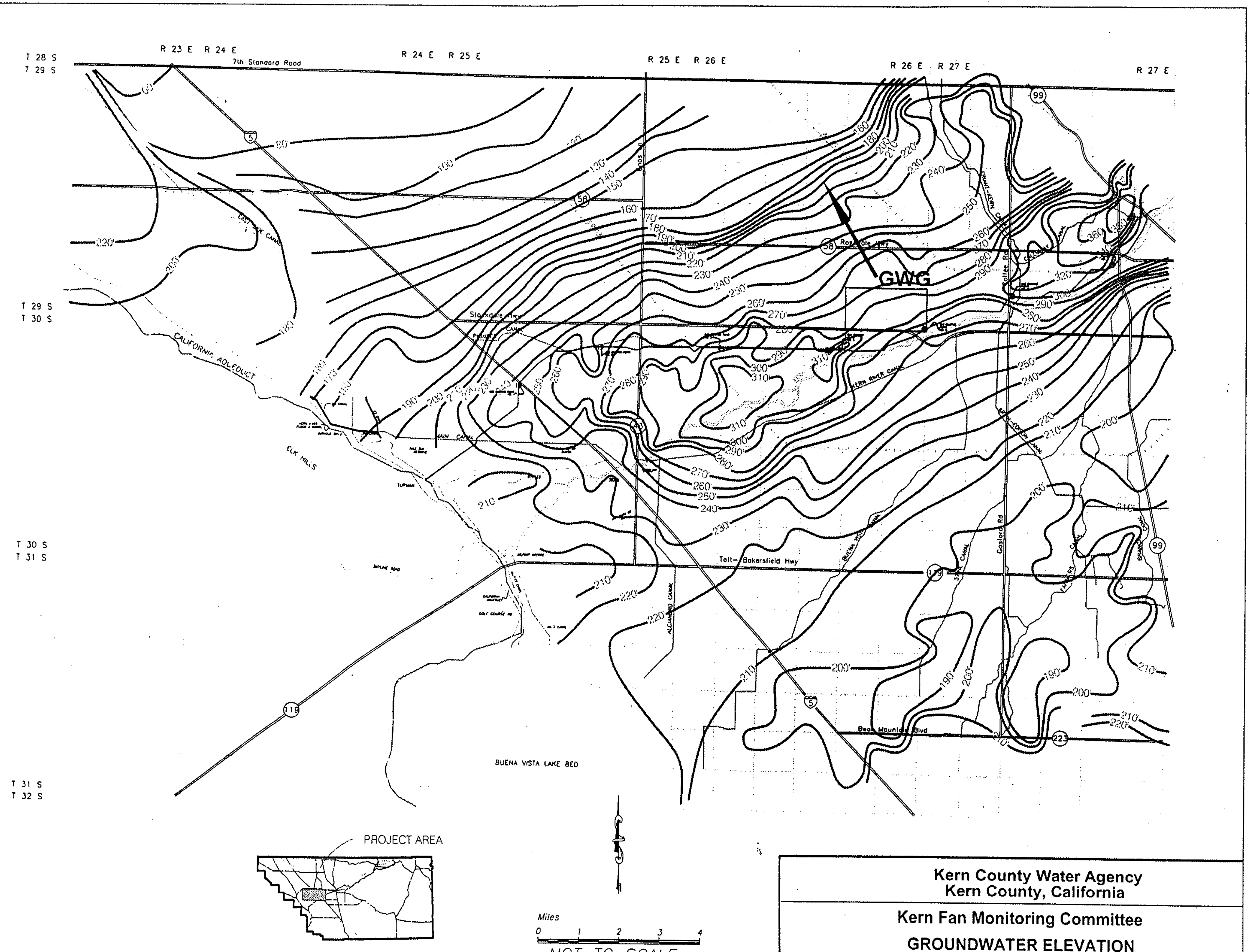


FIGURE 8. KCWA 1998 Groundwater Elevation Map (wet- year gradient).

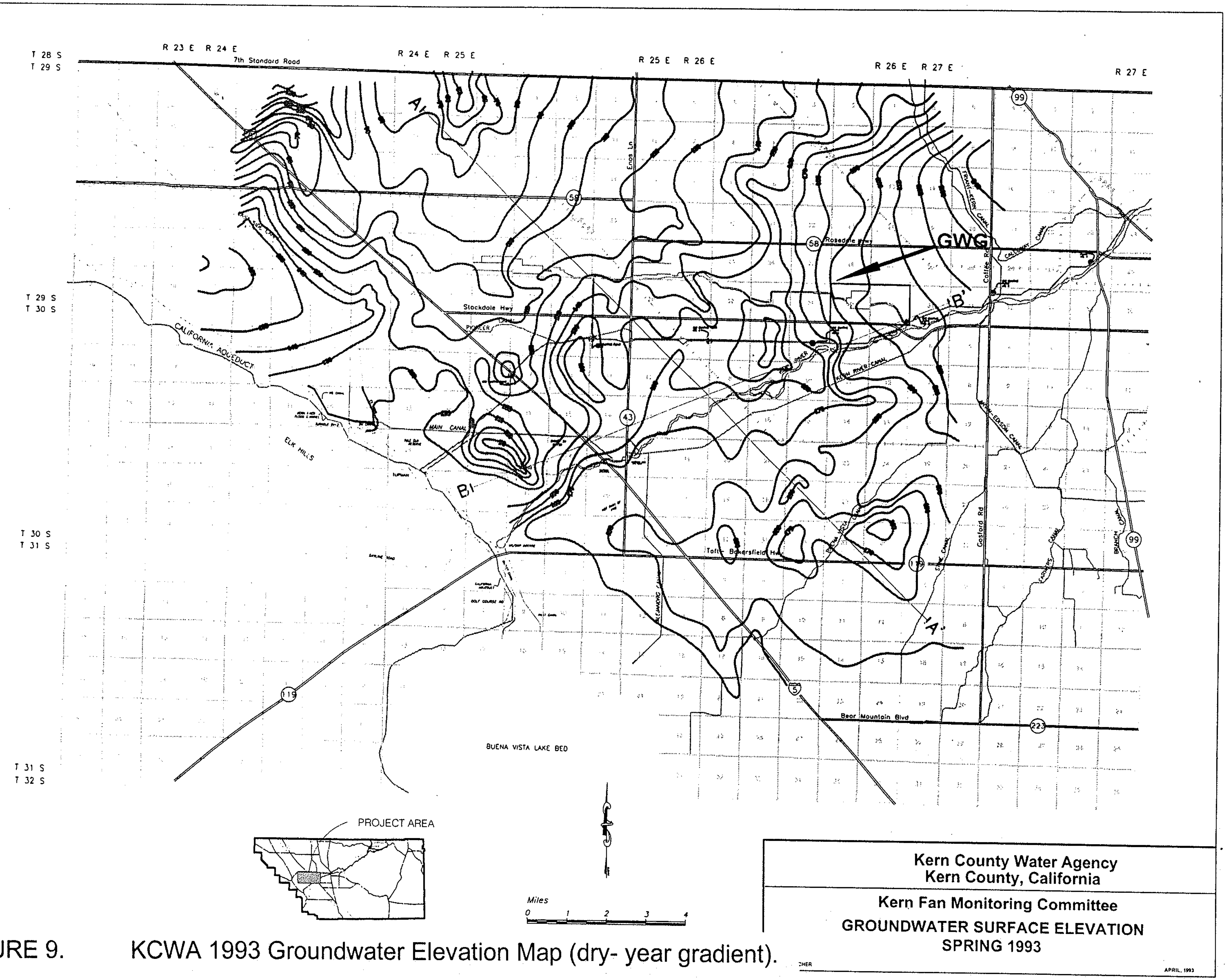


FIGURE 9. KCWA 1993 Groundwater Elevation Map (dry- year gradient).

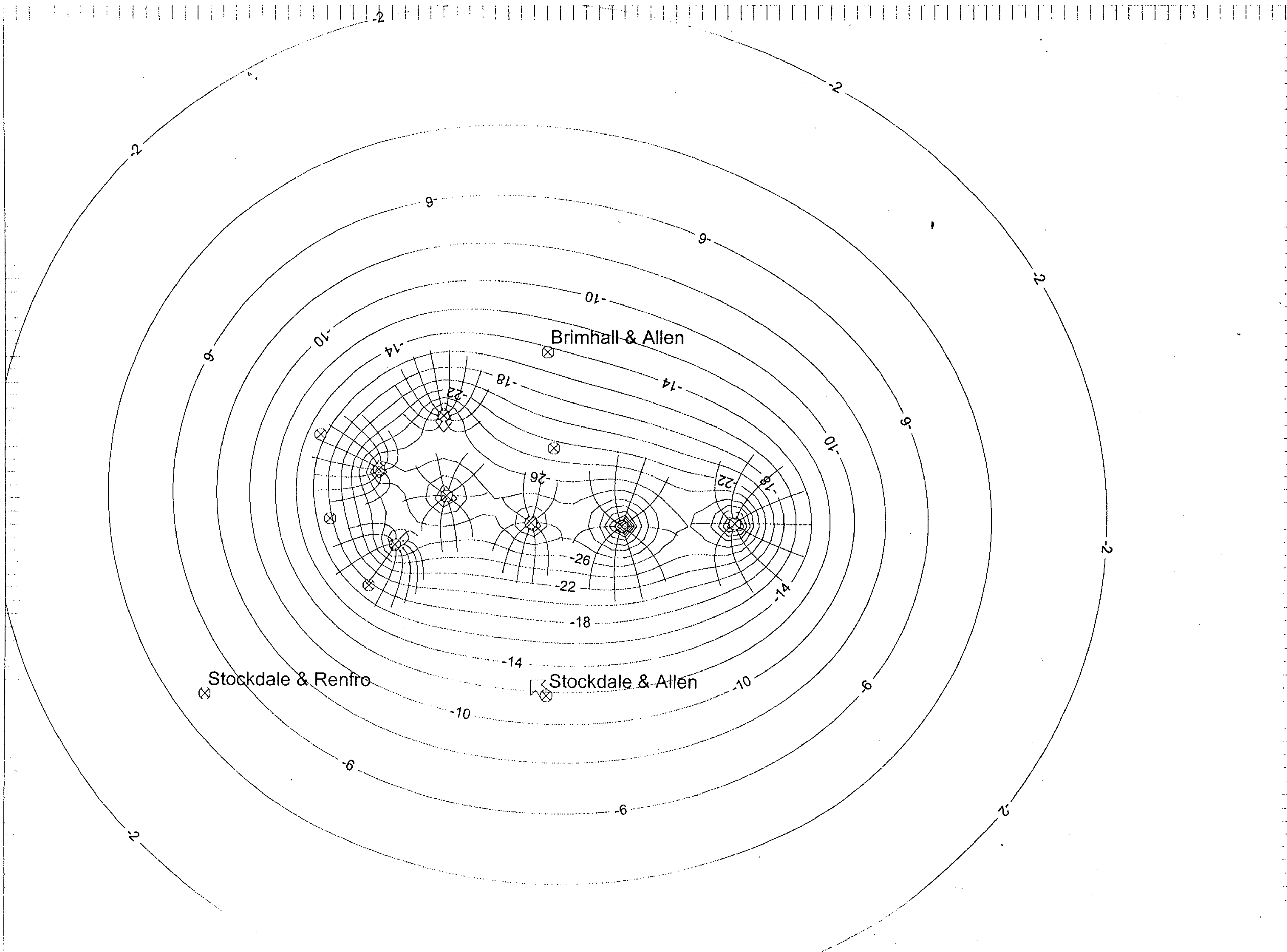


FIGURE 10. Base Case Drawdown Map, leaky aquifer at t = 300d.

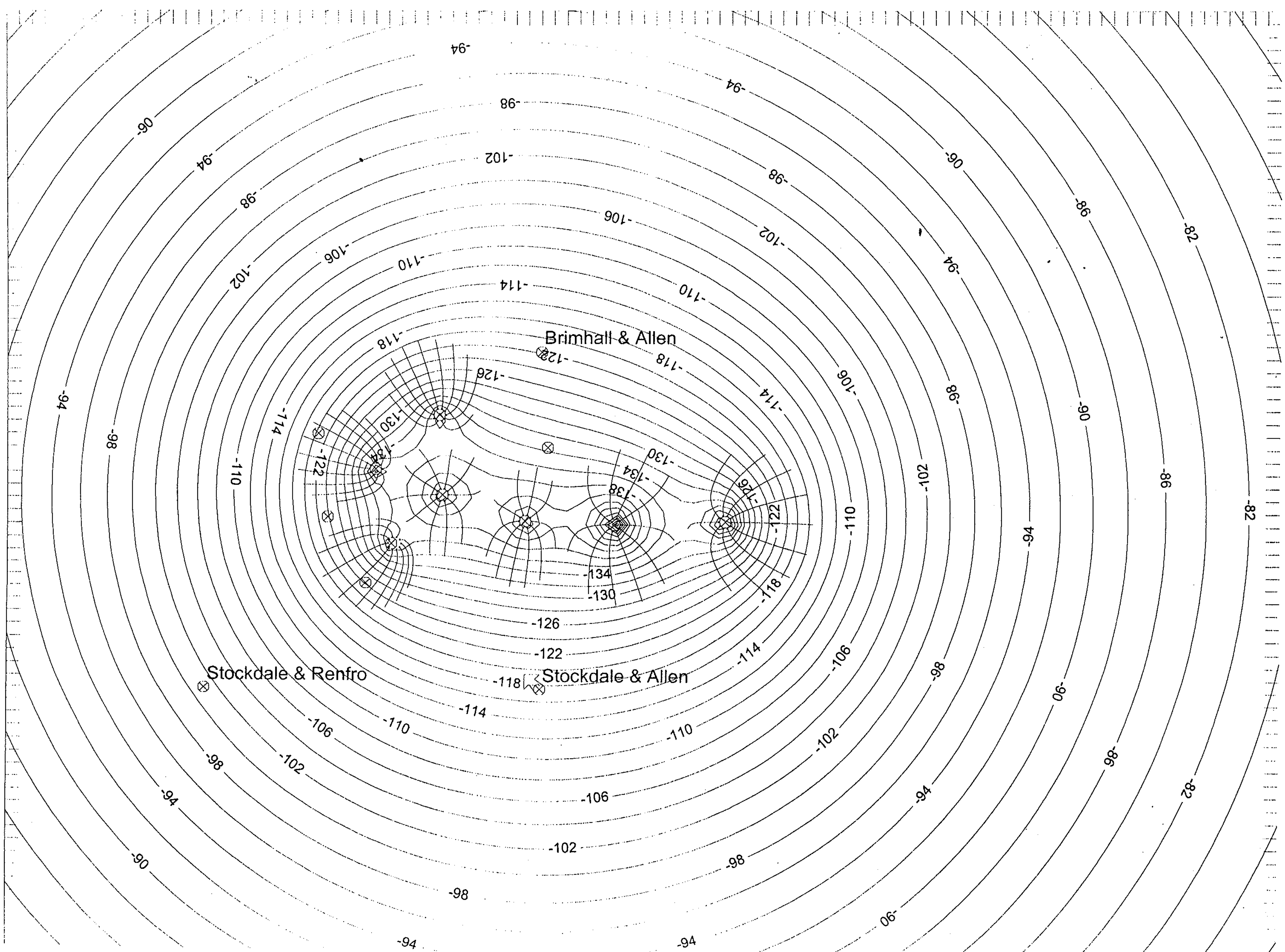


FIGURE 11. Base Case Drawdown Map, confined aquifer at t = 300d.

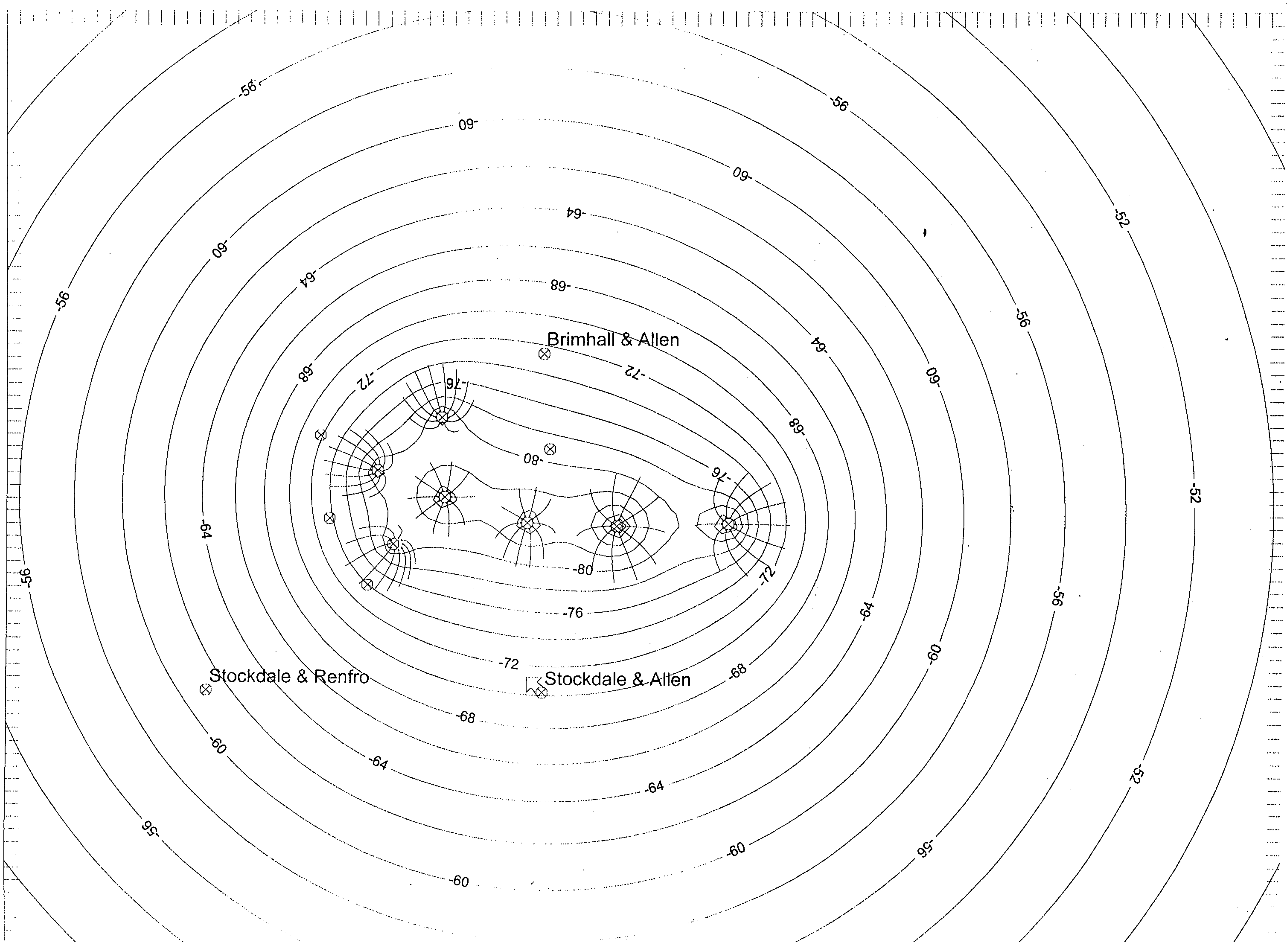


FIGURE 12. Base Case Drawdown Map, unconfined aquifer at t = 300d.

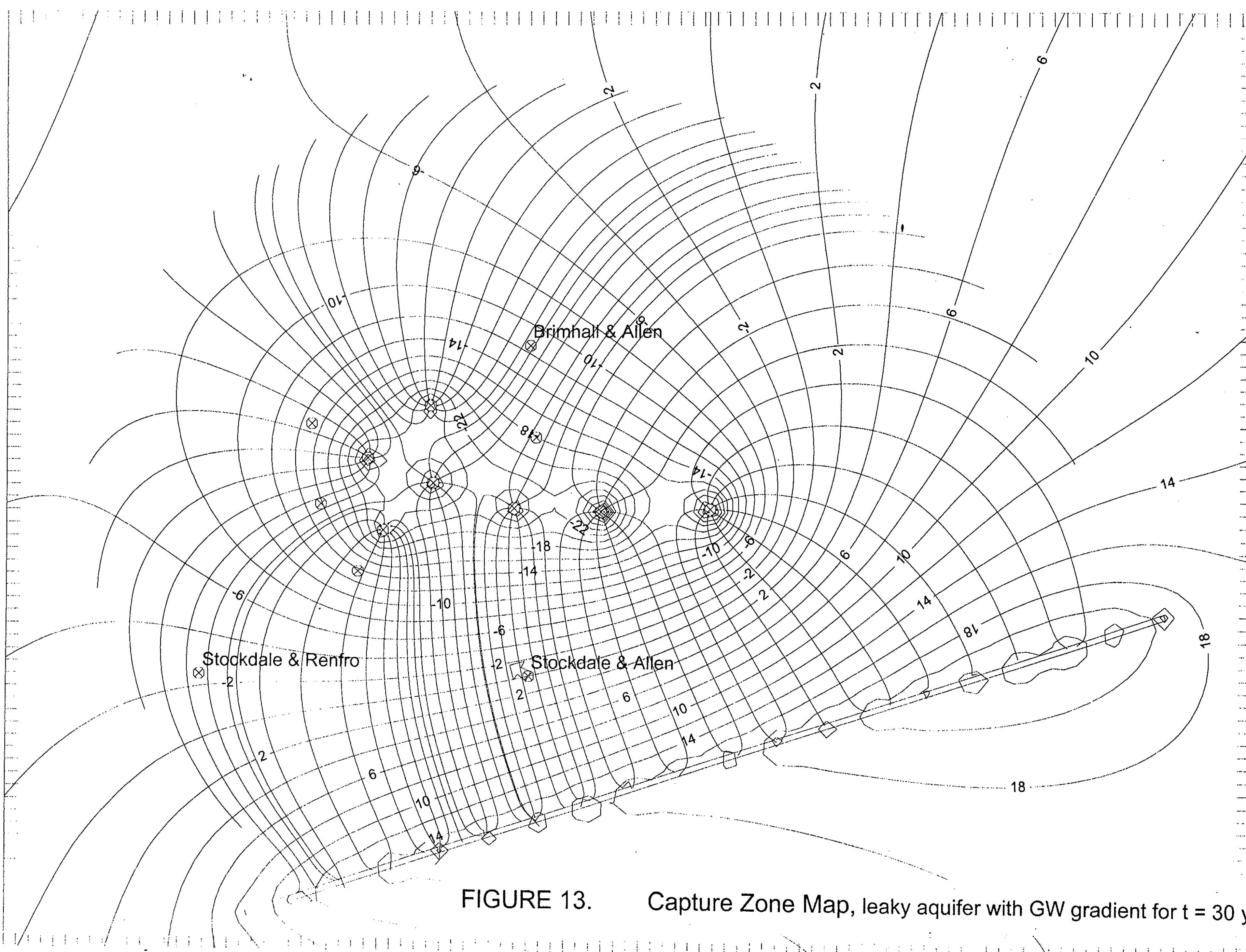


FIGURE 13. Capture Zone Map, leaky aquifer with GW gradient for t = 30 yr.

ID4 / KT / RRB Well Field Predicted Drawdown Summary.

Aquifer Model		pumping time (d)	Predicted drawdown at 1000 ft (ft)	3000 ft (ft)	5000 ft (ft)
300 days pumping					
semi-confined, B = 2600		300 d	12 - 20	4 - 7	1 - 3
semi-confined, B = 3200	*	300 d	16 - 24	6 - 10	3 - 5
semi-confined, B = 6000		300 d	28 - 38	16 - 23	10 - 14
semi-confined, B = 10000		300 d	42 - 52	28 - 36	20 - 26
unconfined, low		300 d	22 - 30	13 - 19	9 - 14
unconfined, high		300 d	71 - 78	62 - 69	56 - 62
confined		300 d	120 - 132	103 - 115	98 - 103
3-years pumping					
semi-confined, B = 3200	*	3 yr	16 - 24	6 - 10	3 - 5
unconfined, high		3 yr	79 - 86	70 - 77	65 - 70
confined		3 yr	134 - 146	118 - 130	105 - 117

* = base case

FIGURE 14. Table of Drawdowns versus Distanc

ID4 / KT / RRB Well Field Predicted Capture Zone Summary.

Aquifer Model	Distance* Downgradient (ft)	Distance* due North (ft)	Distance* due East (ft)	Distance* due South**	Distance* due West (ft)
Pumping Duration (yr).					
1	1100	2300	1600	1500	1300
2	1800	2900	2300	2200	1800
5	2800	4000	3600	4100	2900
10	3700	5000	4800	4000 - 4600	3800
20	4600	6200	6100	4000 - 6200	4600
30	5100	6800	6800	4000 - 6200	5100

* Distance measured from nearest well in well field.
** South-ward limit is at Kern River recharge boundary.

FIGURE 15. Table of Capture Zone Perimeter Distance versus Time.

GROUNDWATER CONDITIONS AND POTENTIAL IMPACTS OF PUMPING FOR THE ID-4 KERN PARKWAY AND ROSEDALE-RIO BRAVO WSD PROJECTS

prepared for
Improvement District No. 4
Kern County Water Agency
Bakersfield, California

by
Kenneth D. Schmidt & Associates
Groundwater Quality Consultants
Fresno, California

January 2003

SUBSURFACE GEOLOGIC CONDITIONS

The project site is on the upper part of the Kern River fan, where coarse-grained deposits are predominant above a depth of about 700 feet. As part of this evaluation, drillers reports and electric logs were obtained for wells and test holes in the vicinity. Two subsurface geologic cross sections were then developed. Figure 1 shows the locations of the River Parkway and proposed RRBWSD wells, the cross sections, and locations of other selected wells referenced in this report. Cross Section A-A' extends generally along the Kern River, from near Heath Road on the southwest, through a number of ID-4 and City of Bakersfield wells, to near the Atchison, Topeka, and Santa Fe Railroad tracks on the northeast. Cross Section B-B' extends from near Palm Avenue north of Brimhall Road on the northwest to the southeast through several KCWA wells, to near Calloway Drive, north of Fraser Road.

Cross Section A-A' (Figure 2) is oriented parallel to the inferred dip of the alluvial deposits. Coarse-grained deposits extend to a depth of at least about 750 feet along much of the section. Stream channel deposits (coarser than sand) are indicated to be present along most of the section. These deposits generally extend to greater depths as one progresses farther southwest. Fine-grained strata that could act as significant confining beds are of limited extent along the section, except below a depth of about 450 to 500 feet. A fairly continuous confining bed (primarily clay) appears to be present below this depth and above a depth of about 750 feet along most of this section. A localized shallow potential confining bed appears to be present primarily west of Calloway Drive, along this section. The top of this layer is about 150 feet deep. The layer appears to be discontinuous, the deposits are not primarily clay, and the bed is indicated to be much less effective than the deeper more extensive confining bed.

Cross Section B-B′ (Figure 3) generally extends perpendicular to the inferred dip of the alluvial deposits. Coarse-grained deposits are also predominant along this section, and are overwhelmingly present in the area north of the Kern River. Stream channel deposits (coarser than sand) are present along this section only in the areas east of Allen Road, and most of these are near or south of the Kern River. A localized, possibly significant shallow confining bed (primarily clay) is present along this section south of the Kern River. The top of this bed is about 150 feet deep. A more extensive deeper confining bed is present below a depth of about 500 feet along the section. This bed is thicker to the southeast, and thinner to the northwest.

Additional subsurface geologic cross sections have been prepared by Environmental Resources Management (2000) for the area east of the Friant-Kern Canal and south of the Calloway Canal.

These sections extend to a depth of about 250 feet and provide more information in the area of volatile aromatic and MTBE-contaminated groundwater, which is discussed in a subsequent section of this report.

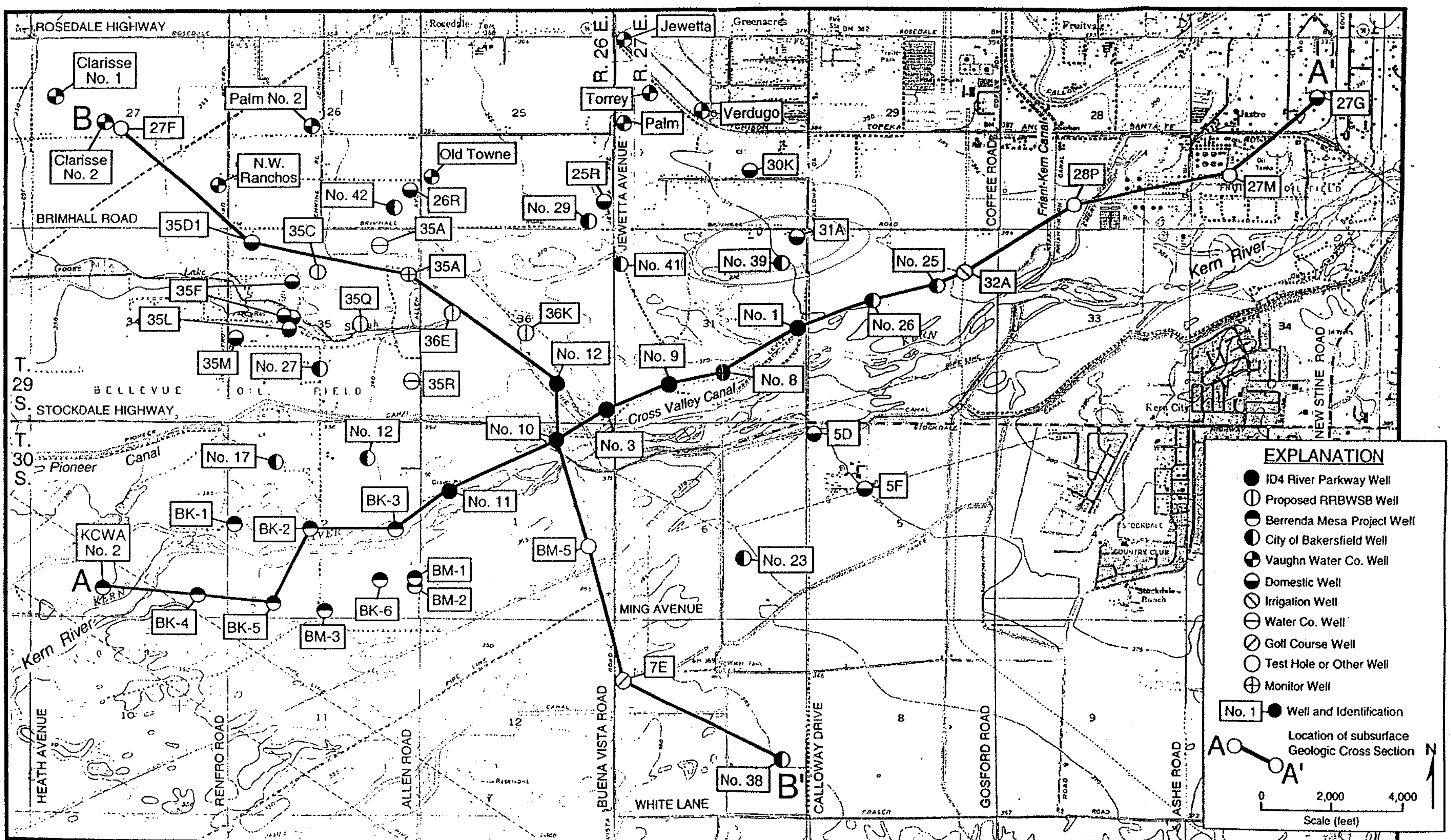


FIGURE 1 - LOCATION OF SELECTED WELLS AND SUBSURFACE GEOLOGIC CROSS SECTIONS

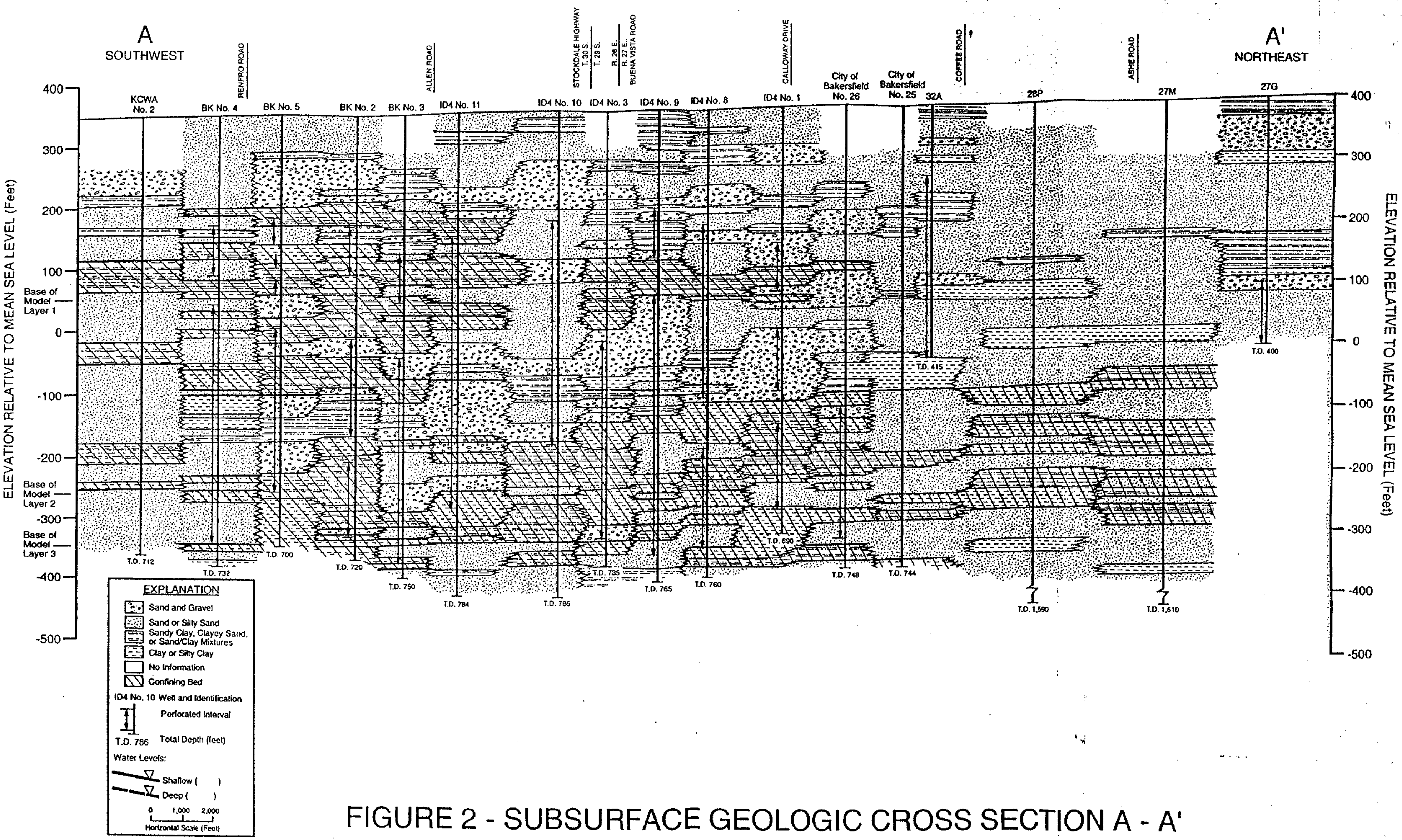


FIGURE 2 - SUBSURFACE GEOLOGIC CROSS SECTION A - A'

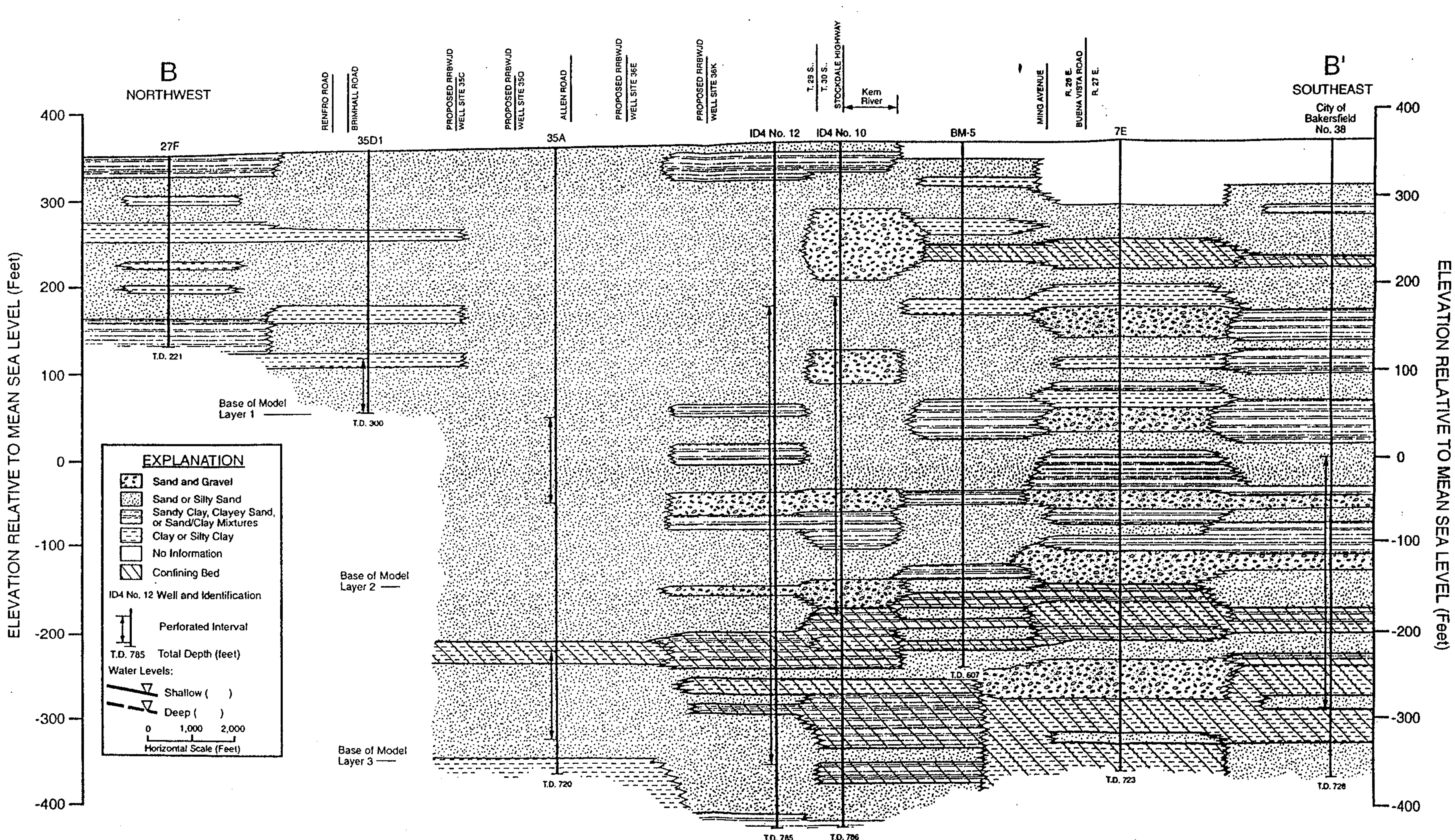


FIGURE 3 - SUBSURFACE GEOLOGIC CROSS SECTION B - B'

AQUIFER CHARACTERISTICS

Aquifer characteristics have previously been determined from the Pioneer and Berrenda Mesa water-banking projects (Kenneth D. Schmidt and Associates, 1998). These, along with the east part of the City of Bakersfield 2,800-acre area, are the closest previously evaluated water banking project areas to the proposed project wells.

The California Department of Water Resources (DWR) divided the alluvial deposits in the Kern Fan area into three layers for groundwater modeling. Layer 1 extends from the land surface to a depth of 300 feet, Layer 2 extends from 300 to 500 feet in depth, and Layer 3 extends from 500 to 700 feet in depth. The DWR provided values of transmissivity for the lower two layers and hydraulic conductivity for the upper layer. However, these values weren't based on aquifer tests and evaluations of some recovery well pumping, nor recharge mound evaluations for the local area following large-scale recharge. Substantial aquifer test data are now available for dozens of water bank project recovery wells, including five of the ID-4 wells. In addition, more information on the upper layer aquifer characteristics is available for 1) aquifer tests when the water level was relatively shallow, and 2) evaluations of water-level rises associated with recharge activities for the water-banking projects.

For the Pioneer and Berrenda Mesa projects, values of aquifer characteristics from the DWR Kern Fan Model for areas close to the proposed project were provided by KDSA (1998, Appendix A). Appendix B of that report contained transmissivity values for Layer 1 and the combined values for all three layers when water levels are shallow. Transmissivity values are normally expected to be higher when water levels are shallower and the saturated thickness of the alluvial deposits is greater. DWR model values were modified to incorporate the results of aquifer tests and mounding evaluations. A significantly higher transmissivity (247,000 gpd per foot) was indicated for the part of the Pioneer area north of the Kern River, compared to the model values. This assumes a starting water level of only 10 feet in depth (the shallow water level condition).

In the Pioneer Project drawdown evaluation, drawdowns were calculated both for a shallow and intermediate starting depth to water. Based on available information, such as specific capacity values for wells covering different time periods, the transmissivity values for the drawdown calculations starting at the intermediate water level (120 feet deep) were reduced only about fifteen percent from those for the shallow water-level conditions (10 feet deep).

Table 4 shows the results of aquifer tests for five ID-4 wells, based on Summer 2001 tests. Pumping rates for these 72-hour constant discharge tests ranged from about 4,500 to 5,055 gpm. Specific capacities ranged from 163 to 232 gpm per foot of drawdown, which are some of the highest observed for such wells in the Kern Fan. The static water levels at the time of these tests averaged about 116 feet deep, or near the "intermediate" level, as previously defined. Transmissivity values for the drawdown measurements were higher than corrected recovery values. Drawdown values for these tests are indicated to be more meaningful, because the duration of measurements was 72 hours, compared to only from

TABLE 4 - RESULTS OF AQUIFER TESTS ON PROJECT SUPPLY WELLS

Well No.	Date	Static Level (ft)	Pumping Rate (gpm)	Pumping Level (ft)	Drawdown (feet)	Specific Capacity (gpm/ft)	Transmissivity (gpd per ft)	
							Drawdown	Recovery
8	6/01	116.6	5,055	144.1	27.5	184	381,000	267,000
9	7/01	116.5	4,980	144.6	28.1	177	346,000	-
10	6/01	114.5	5,000	139.1	24.6	203	440,000	377,000
11	5/01	118.1	4,980	148.6	30.5	163	438,000	227,000
12	8/01	116.4	4,500	135.8	19.4	232	440,000	410,000

Drawdown values are for 72 hours of pumping. Recovery values are for only 3 to 6 hours of recovery. The drawdown values are indicated to be more representative.

three to six hours for the recovery measurements. Previous evaluations have indicated that these wells are highly efficient, thus making the use of drawdown measurements more meaningful. Transmissivities ranged from 346,000 to 440,000 gpd per foot, and averaged 409,000 gpd per foot. Although these values are relatively large compared to those elsewhere in the Kern Fan, they are consistent with the high specific capacities for the ID-4 wells. The average transmissivity value for the tested wells was used for drawdown calculations provided later in this report.

The storage coefficient can't be readily determined from the available pump tests, mainly because the tests could not be run for long enough periods in the absence of interference with other wells. Results of short-term tests on wells tapping layered deposits often provide low values for the storage coefficient, which aren't representative of long-term conditions. The average specific yield of Layer 1 is estimated to be about 17 percent, based on the DWR groundwater modeling. Specific yields for Layers 2 and 3 weren't provided in the modeling reports, because it was assumed that groundwater in these layers is confined (ie. specific yields would not be applicable). The measured water-level declines in KCWA recovery wells during the 1991 recovery pumpage provide the best long-term storage coefficients in the area. The best specific yield value that can be used along with the previously developed values for transmissivity to explain the observed water-level declines due to the 1991 recovery pumpage is 0.10. This is thus considered an appropriate value to use to estimate future water-level declines due to recovery pumpage for the proposed projects. ?

State of California
The Resources Agency
DEPARTMENT OF WATER RESOURCES
San Joaquin District

DEVELOPMENT AND CALIBRATION OF THE KERN FAN GROUND WATER MODEL

by

Robert J. Swartz
Engineering Geologist

Office Report

July 1995

CONCEPTUAL MODEL OF THE KERN FAN GROUND WATER MODEL

The following section describes the physical environment and conceptualized model layout of the study area. Included in this description are discussions of the general geology and movement of ground water in the study area. Additionally, the physical framework and processes which impact the modeled system are discussed.

Physical Setting

Understanding the physical setting is an important first step in model development. Included in the physical setting discussion of KFGWM are the general geology and ground water.

Geology

The KFE is located in the southern San Joaquin Valley, a large northwest-trending, asymmetrical trough. Overlying the crystalline basement of the Valley are more than 15,000 feet of shales, siltstones, and sandstones deposited in a variety of marine environments. The upper 3,000 to 4,000 feet of these sediments were deposited primarily in alluvial fan, lacustrine, or deltaic environments (Wilson, 1993). ?? marine?

The upper several hundred feet of these deposits are dominated by unconsolidated sediments comprising the upper portion of the Kern River Alluvial Fan (Kern Fan) as described by Dale et al. (1966) (Figure 1). The Kern Fan comprises some 800 square miles of surface area and contains the principal water-bearing sediments of the aquifer. Sedimentary deposits in the fan are highly heterogeneous, with a predominance of sand and gravel deposited in channels and finer-grained overbank deposits. Sediments of the Kern Fan are derived from weathered granodiorite of the Sierra Nevada Range which is transported into the southern Valley by the Kern River.

The KFE, the core of the study area encompassed by the KFGWM, is located on the distal portion of the Kern Fan and straddles the Kern River channel (Figures 1 and 2). The distal portion of the fan also contains lower energy deposits, such as clays in the Buena Vista lacustrine basin to the south, and several laterally discontinuous clay/silt layers.

The Elk Hills, located at the western limit of the Kern Fan (Figure 1), are composed of three major doubly-plunging anticlines (Reid, 1990). The consolidated sedimentary rocks of this structure act as a physical barrier to ground water movement to the west of the study area.

Ground Water

Many sources contribute to ground water recharge. Historically, the Kern River has been the primary source of direct recharge to the aquifer. Infiltration of excess irrigation water is also a significant component of recharge. Precipitation is a negligible source of recharge because the region gets fewer than 6 inches of annual rainfall.

While the Elk Hills act as a barrier to ground water flow to the west, there are no significant faults or barriers to lateral movement of water within the study area. One minor permeability barrier exists along the western margin of the KFGWM. This area trends parallel to the Elk Hills and causes elevated heads in a small area on the western KFGWM boundary.

Laterally discontinuous clay/silt bodies in the study area act as aquitards to vertical flow. A network of multiple-completion monitoring wells on the KFE indicates that these aquitards cause semi-confined conditions in the deeper portions of the aquifer, while the upper few hundred feet of the aquifer is predominantly unconfined. This fact is demonstrated in a hydrograph from a typical study area monitoring well (Figure 3). During the summer months, the two deeper zones tend to diverge from the shallow zone and quickly recover during periods of reduced pumping in winter months.

?

Ground water elevation contours from spring 1993 indicate that the principal direction of ground water flow is from east to west along the Kern River channel. Near the western extent of the study area, flow is naturally diverted by the Elk Hills toward the northwest and, to a lesser degree, toward the south. Historical records indicate this flow pattern has been consistent since at least the beginning of this century.

Conceptual Model Layout

Model conceptualization is a key step in the development of the ground water model. Components of the hydrogeologic setting are transformed into the model's physical framework during the conceptualization process; e.g., the physical structure of the Elk Hills is considered as a no-flow boundary to the southwest of the model. A grid is devised to represent the study area. The size of the rows and columns is based upon the required level of detail for the modeling effort. Also considered in the conceptualization process is the assignment of factors (system stresses) that influence the modeled system.

Physical Framework

Although the depositional system is highly heterogeneous, ground water flow in the aquifer is considered to be isotropic (similar in all directions). A pattern of increasing confinement with

?

depth also exists in the model area. To represent increasing confinement, the KFGWM is composed of three discrete model layers: Layer 1 (ground surface to sea level)[1], Layer 2 (sea level to -200 feet), and Layer 3 (-200 to -400 feet). Layer 1 represents the unconfined portion of the aquifer; Layers 2 and 3 represent semi-confined portions (Figure 4). In general, these layers correspond to perforated intervals of the monitoring well network.

The KFGWM grid is presented in Figure 5. The KFGWM grid is composed of 58 columns and 41 rows, and consists of 2,051 active cells per layer (6,153 total cells). The cells range in size from 160 acres in the northwestern, northeastern, and southeastern corners, to 40 acres in the model's core area. The irregular boundary in the southwest area represents the base of the Elk Hills. Since the KFGWM was developed to simulate the effects of ground water banking activities within the KFE, the remaining boundaries were set approximately 3 to 5 miles away from the KFE's perimeter.

System Stresses

Major internal stresses within the KFGWM include pumpage of ground water for agricultural usage and minor municipal and industrial uses, and ground water recharge resulting from intentional recharge or deep percolation of applied water to crops. Calculation of pumpage and recharge for the KFGWM is an important step prior to running the ground water model.

Model boundaries represent a source of external stresses to the model area. Use of a constant head boundary allows water to flow in and out of the modeled area. When ground water levels in perimeter cells are higher than the constant boundary heads, water flows out of the model. When the heads in perimeter cells dip below the constant boundary heads, water flows into the model area.

Each KFGWM cell represents a potential source of pumpage or recharge depending on the activities at or near the location represented by the cell. To simplify the assignment of internal system stresses, cells were grouped into hydrologic zones of similar characteristics of water use and supply. The result is an array in which each active cell is represented by a number corresponding to an identified hydrologic zone (see Appendix A for an example of a pumpage/recharge array). Within each zone, land use data provide individual crop acreages from which water demand is derived.

[1]The KFGWM was originally developed as a four-layer model; however, initial attempts at calibration using MODFLOW's original Block-Centered Flow package resulted in the "drying out" of upper layer cells which rendered them inactive. This problem was eliminated by combining the two uppermost layers into Layer 1.

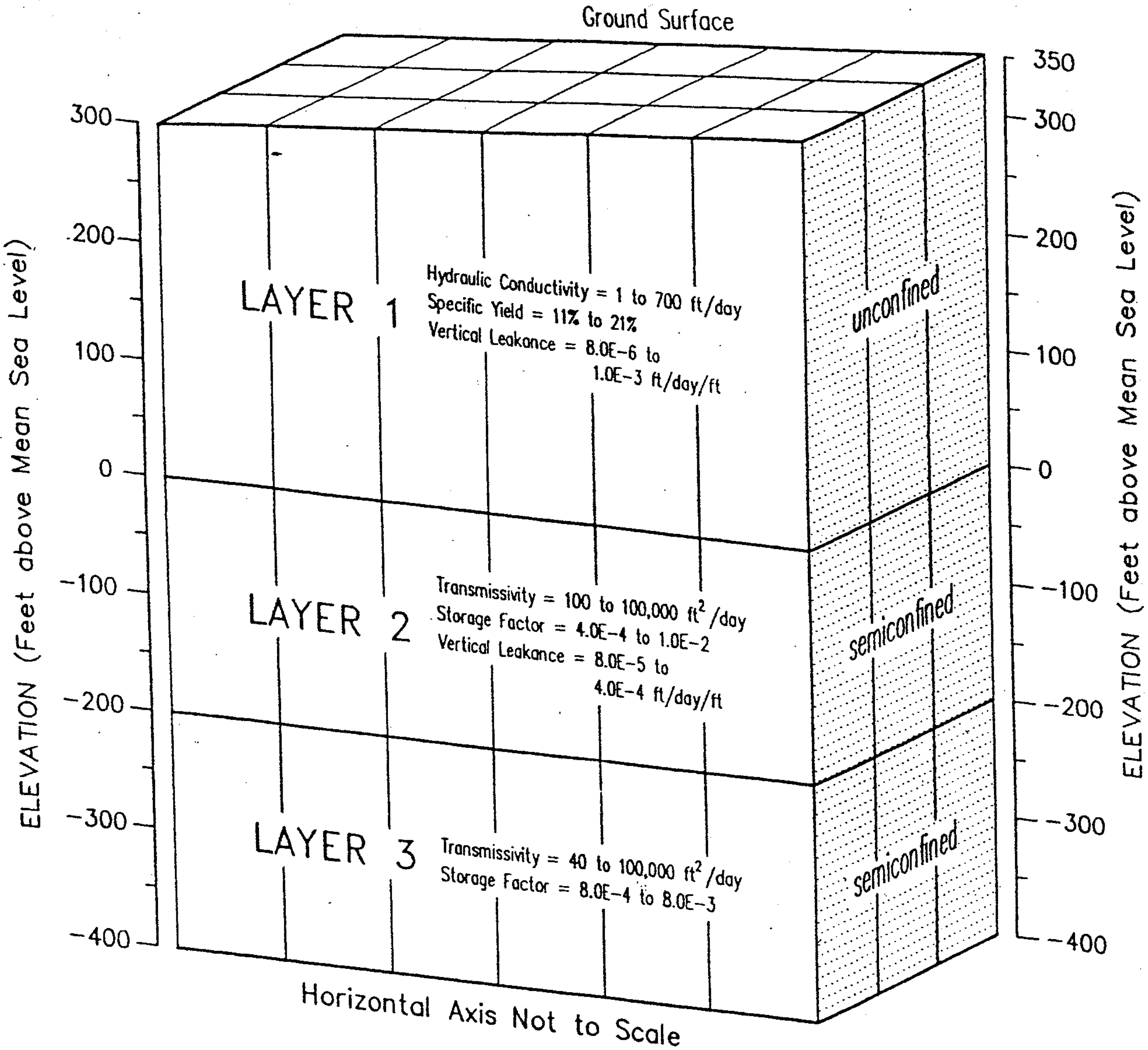


Figure 4. Conceptualized Layers and Ranges of Hydrogeologic Parameters in KFGWM

Referenced from page 16.

DEVELOPMENT OF HYDROGEOLOGICAL PARAMETERS

Over 1,000 lithologic and electric logs from wells and borings drilled within the project area were collected and evaluated for reliability and detail of information. Each log was assigned a category from A through F. The A-log category was assigned to geologist logs with detailed lithologic descriptions. The B-logs are driller logs with detailed descriptions. C-logs are driller logs with some detail, and D-logs are driller logs with little detail. E-logs are electric logs. Every available E-log was used in the analysis. F-logs are driller logs which are too generalized to adequately represent the subsurface environment and were not used in the analysis.

Each evaluated log was plotted spatially on a KFGWM map, then two to three logs per section (1 square mile) were chosen to represent the geology of that section. The decision of which logs to accept was primarily based on obtaining proper well density, followed by the category assigned to each log. Two lists of wells were compiled for further analysis: (1) wells representing shallow depths (ground surface to sea level), and (2) wells representing deep units (sea level to 400 feet below sea level). The logs were then utilized to develop estimates of aquifer hydrogeological parameters.

A guess based on E-log/grain size

But it doesn't mean it's accurate.

Specific Yield

Specific yield (Sy) is a unitless number representing the ratio of the volume of water which gravity drains from a porous material to the total volume of that porous material (Fetter, 1988). Each lithologic interval from the well logs was assigned a value of Sy based on the sediment type in that interval (Table C-1). The basis for use of this method is well documented (DWR, 1961; Johnson, 1967; Walton, 1970). The lithologic interval and corresponding Sy value for each well were entered into a data file (kfespyd.god) for processing by a DWR FORTRAN program SPYD, which computes a weighted average of Sy for 100-foot intervals in each well. Each 100-foot interval was subsequently averaged into intervals (utilizing a spreadsheet) representing the KFGWM layers. The upper layer (Layer 1) utilized an average of the uppermost three 100-foot intervals. KFGWM Layer 2 was represented by the fourth and fifth 100-foot intervals, and KFGWM Layer 3 was represented by the sixth and seventh 100-foot intervals.

mostly $15 \leq S_y \leq 19\%$ for Kern Fan Area.

TABLE C-1

ASSIGNED Sy VALUES TO
AQUIFER SEDIMENT TYPES

Soil Type	Sy Percent
SILT (?)	?
Clay	3
Sandy clay	6
Clay with streaks of sand (%?)	6
Sand with streaks of clay (")	12
Fine sand	20
Medium sand	27
Coarse sand	30
Medium sand with gravel	20
Coarse sand with clay	12
Gravel	30
Medium to coarse sand	27
Medium sand with streaks of fine sand	17
Coarse sand and gravel	27
Sand (fine to medium)	20
Silty sand	17

?? ? ?

why not LSQFIT?

These are not soils

Hydraulic Conductivity

Hydraulic conductivity (K) represents the rate at which water moves through a porous medium (expressed in KFGWM as feet/day). Values for K based on soil type were obtained from published correlation charts (Driscoll, 1986). Although a range of values is given for each soil type in the Driscoll chart, a typical K value was selected based on first-hand knowledge of the soils within the southern San Joaquin Valley. These estimated values of K (for various soil types) were then plotted against the calculated values of Sy (Figure C-1). The points on the graph represent corresponding values of Sy and K based on soil type; Line 1 is a visually fit line of these points and was subsequently used to estimate all values for K based on the calculated values of Sy for each well. However, the estimates of K yielded modeled heads below the observed heads within the aquifer

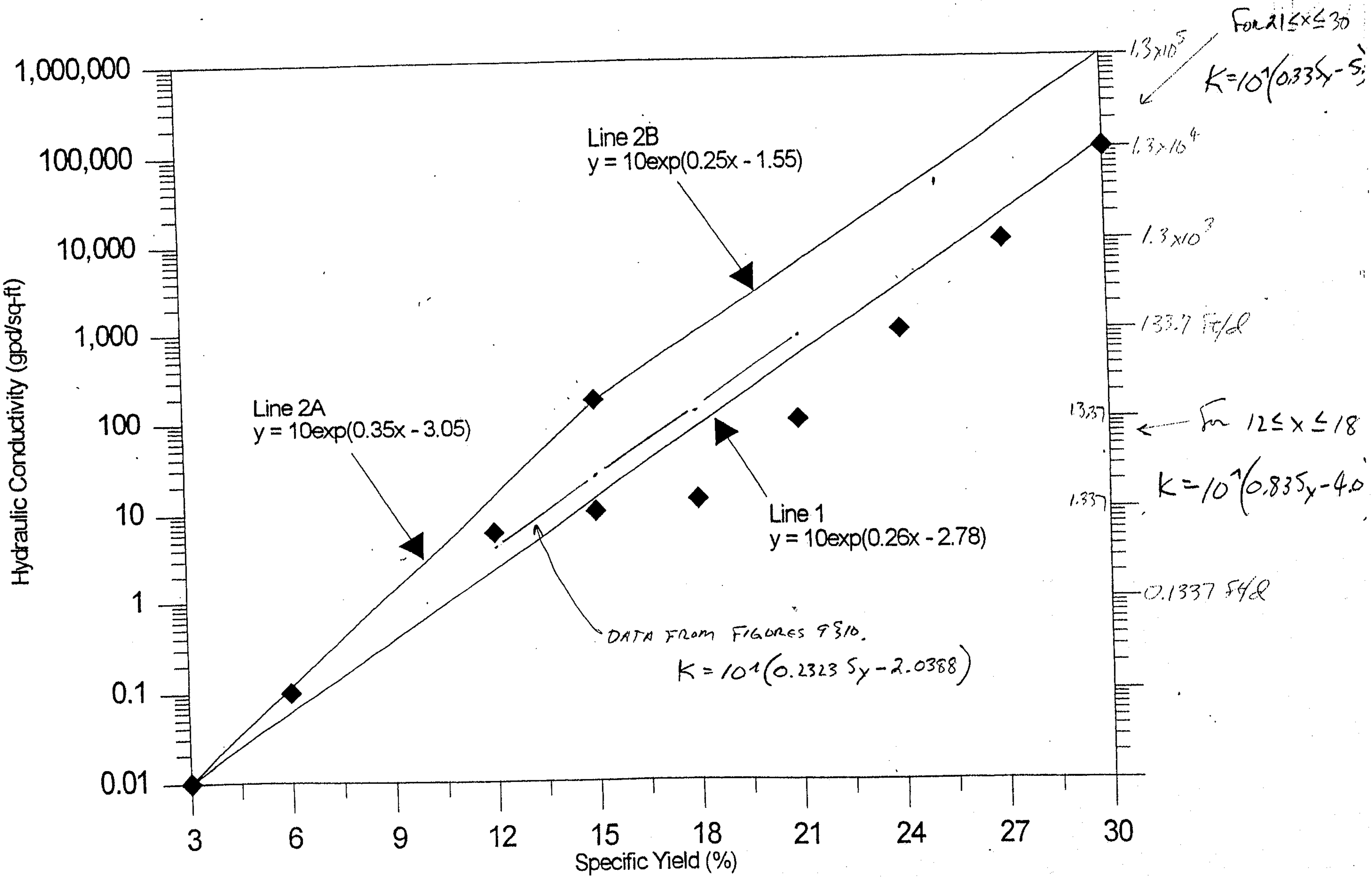


Figure C-1. Graph for Estimating Hydraulic Conductivity from Specific Yield.

during an initial model demonstration run. Values of K were raised in order to compensate for the low head values.

? - it's a log plot! THIS HAS NOTHING TO DO WITH REAL MEASUREMENTS ON REAL SEDIMENTS. THIS IS FICTITIOUS MODEL DATA.

A second linear relationship between Sy and K (Lines 2A and 2B) was developed based on the premise that the maximum value of K used previously was not high enough to represent the coarse sediments within the basin; a higher K value is further supported by the lack of significant, laterally-continuous clay layers within the project area. The "kink" point on the line at which the slope changes is an estimated value of K based on limited pump recovery test data from the City of Bakersfield's 2,800-Acre Recharge Site (Wilson, 1989). Equations to fit this line were calculated and used in model calibration. The result was a good comparison between modeled piezometric surface and observed piezometric surface for the calibration years from 1988-1991.

3 YEARS OF DROUGHT, i.e. CALIBRATION IN YEARS OF MOSTLY VERTICAL DROP IN WT & NOT MUCH HORIZONTAL FLOW.

Storativity

surprise!!

Storativity (S) is a dimensionless coefficient defined as the volume of water stored or expelled per unit area of an aquifer per unit change in head (Fetter, 1988). Values for S were also estimated using a graphical method developed by plotting Sy versus S (Figure C-2); the initial values of S were obtained from published data (Driscoll, 1986) and estimated degree of confinement (Table C-2). p118 Line 1 represents the first estimated set of values for S. However, these values did not fit well into the initial model calibration. The maximum S value was increased for Line 2 because the aquifer appears to be lacking truly confined zones. Line 2 was successfully used to estimate values of S in subsequent model calibration runs. The S value obtained from the graph is for a semi-confined aquifer; values of S for a confined aquifer were estimated by decreasing the value obtained for S from the graph by a factor of 10. ??

This is what happens when you don't have any real data!

Vertical Hydraulic Conductivity

Vertical hydraulic conductivity (Vcont), as defined in MODFLOW, is vertical K divided by the distance from the center of one layer to the center of the layer below it. This parameter was difficult to estimate using the average Sy values, because a thin clay layer could potentially represent the most restrictive unit to vertical flow; this would not present a significant restriction to flow in the horizontal direction. Values for Vcont were initially estimated as approximately 0.0004 feet/day. This parameter was then modified as necessary during the calibration process.

WHY DOES THE DWR THINK these parameters CORRELATE AT ALL?

K ÷ ST distance gives a parameter in reciprocal time.
We call this "Leakance", not Kvert.
The Darcy flux is Leakance times head difference.

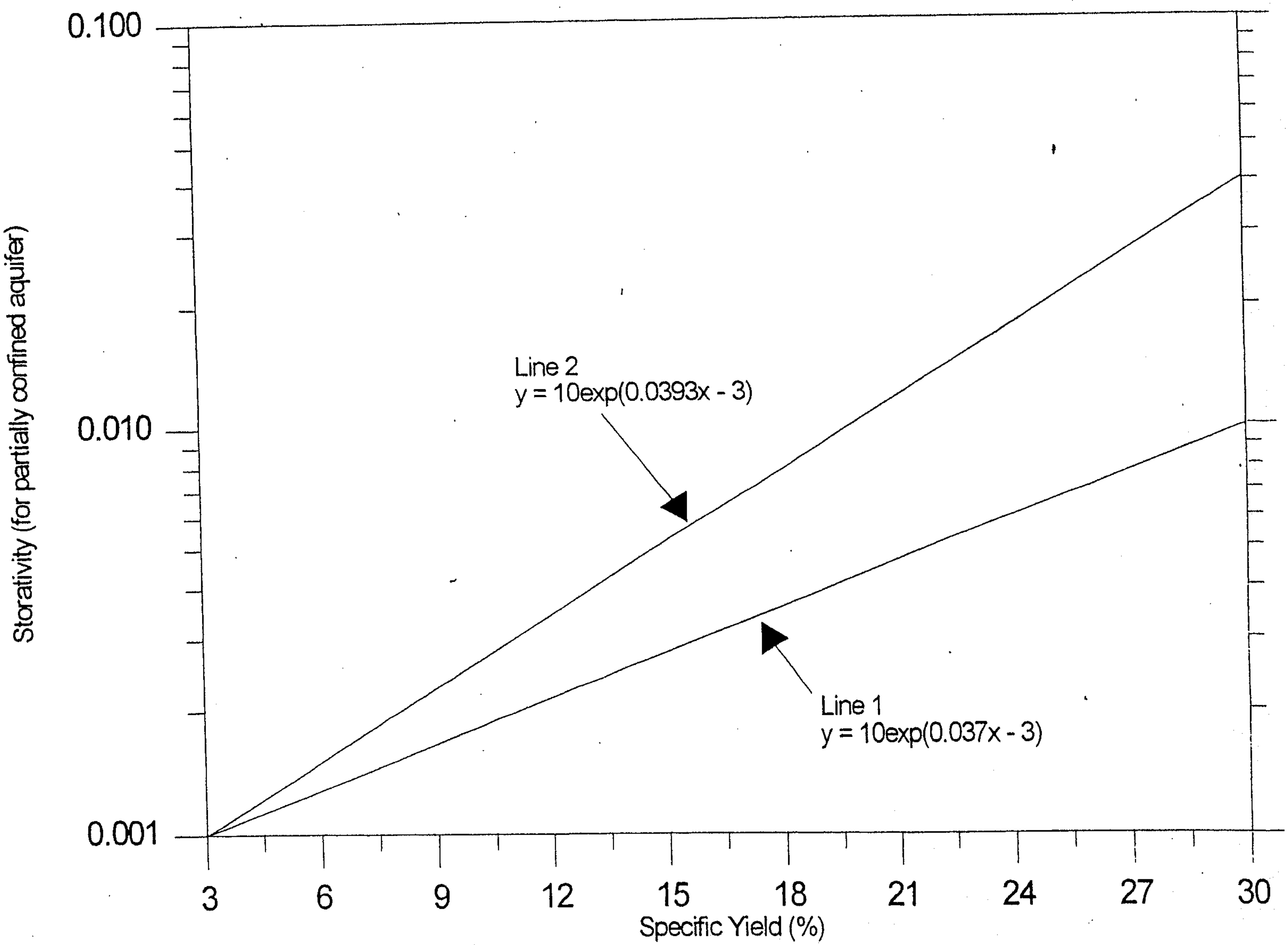


Figure C-2. Graph for Estimating Storativity from Specific Yield.

TABLE C-2

RANGE OF STORATIVITY (S) BASED ON SOIL TYPE AND DEGREE OF CONFINEMENT

Layer	Clay	Sand
1	0.03	0.30
2	0.0001	0.004
3	0.0001	0.004

Appendix 2.

Mathematical Aquifer Models.

Appendix 2.
Mathematical Aquifer Models.

Aquifer behavior. An aquifer is a porous medium consisting of one or more layers of rock or sediment which can store and transmit water in useful quantities. In the simplest terms, ground water aquifers function in two ways: aquifers function as a reservoir to store water and aquifers function as a pathway for ground water flow. All changes in aquifer storage or aquifer flow are caused by either gains or losses of water from the aquifer due to any of several natural or manmade actions.

In the case of aquifer storage, hydrologists evaluate ground water storage with a map of the water level elevation which basically represents how "full" the aquifer is at any particular location and time. If a hydrologist wants to determine the hypothetical impacts of gaining or losing water from the aquifer due to, for example, recharge ponds or pumping wells, then the impacts would be represented by changes in the configuration of the water table as presented in one or more maps or cross sections. All estimates of the change in aquifer storage use the area-weighted vertical changes in this surface to calculate the volumetric change in storage.

In the case of aquifer flow, hydrologists evaluate ground water flow in the aquifer by determining the flow paths (which we call particle trajectories) and flow rates (particle velocities) that describe the movement of water molecules in the aquifer. If a hydrologist wants to determine the hypothetical impacts of changing the aquifer dynamics due to, for example, recharge ponds or pumping wells, then the impacts would be represented by changes in the lengths and directions of the flow paths as presented in one or more maps or cross sections.

Impact analysis. Hydrologists use mathematical aquifer models (sets of equations including sets of conditions and parameters) to calculate the hypothetical water table elevation maps and the ground water flow path maps which are predicted to result from the changes in aquifer dynamics due to recharge ponds, pumping wells, and/or any other natural or manmade action of interest. Let us consider the potential water table drawdown and inward radial flow of ground water due to installing and then pumping a new water well. Let us refer to our evaluation as an impact analysis. Our desired output is a map which shows the hypothetical water table drawdown and ground water flow paths within the capture zone surrounding the new well.

There are many computation methods for predicting drawdown from a pumping well in space and time and every method requires that the user select the equations which are most appropriate for the user's preferred model of the aquifer. In essence, the user must try to select the set of mathematical expressions which best represents the user's physical model of the aquifer. The hydrologist's physical model of the aquifer includes knowledge of the geology and hydrology including the layering, structure, depths, dimensions and physical properties of the aquifer as well as the locations and flow rates of all sources of inflow and outflow to the aquifer such as wells, streams, ponds, etc.

The calculated result, if done correctly, always represents the workings of the mathematical equations but only represents the behavior of the real aquifer to the extent that the parameters, simplifications and assumptions of the mathematical model reflect the true workings of nature. The selection of the mathematical model, the equations, the accuracy of the parameter values, and the representativeness of the calculated output all reflect the experience, expertise, correctness of- and uncertainty in- the judgments of the hydrologist. These judgments cannot be made by the computer and the two main judgments include the choice of mathematical model and the choice of aquifer parameters. There is no such thing as a simple calculation. A good impact analysis rests at least as much on a hydrologist's competence in understanding equations, validity tests, boundary conditions, and model parameterization as it does on the determination of aquifer properties. In our opinion, many hydrologists and engineers who use mathematical models to compute aquifer impacts would benefit from a better background and understanding of the proper use and pitfalls of such models.

<u>Analytical Models.</u> For any scope of work, there are two basic choices of mathematical model. The first choice is to select a "canned" analytical model which best approximates the interpreted aquifer conditions and then supply the user's best estimates of the required aquifer parameter values. The great advantage to this alternative is that the models are fast, convergent, easy to customize and operate, and the models result in a *unique* set of solutions because the degrees of freedom in the model are the same as the number of available parameters. The disadvantage is that the mathematical model may not represent all of the

known or suspected complexities of the real aquifer and the user must evaluate the relevance and magnitude of the possible errors in the results due to the simplifications in the mathematical model. The analytical models which are frequently used today include the familiar equations attributed to Theis, Cooper - Jacob, Hantush, Hantush - Jacob, Neuman, Strack, etc., for all of the useful recharge and recovery interactions (wells, ponds, rivers, surface recharge, etc) for transient and steady- state conditions in unconfined, confined, and leaky aquifers. SSS selected this option for the ID4 project scope of work.

Numerical Models. The second choice, which SSS did not choose, is to design and program a numerical computer model which best approximates the interpreted aquifer conditions in all its 3-D detail and then supply the user's best estimates of the required aquifer parameter values. The only advantage to this alternative is that the model may be designed to any degree of complexity in order to approximate the true aquifer structure. The disadvantages are numerous and punishing. The models are tedious and difficult to build; the models require an impossibly vast knowledge of the aquifer properties because the user must define the value of every aquifer parameter at every depth at every location; the hundreds or thousands of degrees of freedom always outnumbers the amount of real data which causes non-uniqueness[1] and equivalence[2] in the model outputs; and there is a significant likelihood that numerical complexity does nothing to improve the quality or accuracy of the output of the calculation while giving a false sense of precision in the effort. One of the most popular numerical models is actually a number of programs which are all referred to by the name *Modflow* (a trademark of the United States Geological Survey), which are based on a publicly- available computer code developed by the U.S.G.S. and commercialized in several proprietary forms by different scientific software companies. Sierra Scientific Services owns a complete set of Modflow simulators for groundwater flow, contaminant transport modeling, and parameter estimation but SSS favored the analytic model to be better suited to the ID4 project scope of work.

---

[1]Uniqueness is a property of a model solution which refers to the condition that a given set of inputs can result in only a single, fully determined output. Non-uniqueness may refer to a condition in which a given set of inputs may result in more than one fully- determined possible output.

[2] Equivalence is a property of a model solution which refers to the condition that two or more different set of inputs can result in exactly the same, fully determined output.

Appendix 3.

Aquifer Parameters and Parameter Values.

Appendix 3.

Aquifer Parameters and Parameter Values.

The aquifer parameters of interest for mathematical modeling include those *intrinsic* physical properties of the porous media which determine the volume- specific storage and unit flow properties of the aquifer. These intrinsic properties are then combined with the physical dimensions (depths, thicknesses, and gradients) of the aquifer media to determine the full-aquifer behavior. The storage properties include the specific storage (S_s)and specific yield (S_y) of each of the porous media. The required flow properties include the hydraulic conductivity (K), porosity (ϕ), and dispersivity (α) of each of the porous media. The hydraulic conductivity is required for volumetric flux and flow rate in directions of interest (K_h for horizontal flow and K_v for vertical flow), the porosity is required for particle tracking, and dispersivity (both longitudinal α_L and transverse α_T) is required for contaminant transport.

These properties are normally determined either by physical properties measurements on actual rock or sediment samples or by special types of pumping tests on water wells which have been completed across the thickness of the aquifer. Some of these properties vary by several orders of magnitude for common aquifer rock- and sediment- types, so for aquifer materials which have not been measured or tested, there is little likelihood that a best- guess "textbook" value which is based on rock type or another index property will be very close to the actual value. We recommend that the careful determination of the relevant physical properties be an essential and early part of any groundwater program.

It is important to emphasize that the values of these physical properties are all constants for each of the respective aquifer media and that they do not vary with changes in either the water table, or in the pump rate or completion interval of a well, or with any other observed variable, apart from the natural variability of the property within the porous medium itself. It is good practice to measure these properties as many times as possible to determine the average value and range of natural variability for each. And since hydrologists recognize that the natural variability of these parameters may be large, it is best to obtain measured values which are representative of the aquifer under the entire area of interest for which impact analyses are desired, and measured in ways which minimize the unassociated variance in the determination. It is also important to emphasize that few of these properties can be determined directly from well tests and must instead be derived indirectly from well test data by also using other information. The ability to do this is governed by cost, access to wells, and the expertise of the hydrologist to perform the right test and to make the necessary corrections for factors and interferences which otherwise cause errors in the values.

Storage properties.

Water is stored in an aquifer by occupying the intergranular void spaces of the porous aquifer material. The physical amount of water which can be stored in and recovered from a porous medium is the sum of two components; the fillable void space remaining in a rock or sediment which is at residual saturation, and a very much smaller component which is a result of the minute elastic dilation of this void space when the water in the aquifer is under pressure combined with the slight compression of the water itself.

When water is released or recovered from an aquifer, the first water recovered is always that which is released due to the elastic rebound of the pore space and the water. The last water recovered is always that which drains from the pore space and dewaters the aquifer. When water is stored in the aquifer, the reverse actions occur, i.e., water first fills the void spaces and then dilates the void space as the pressure increases.

Specific yield. The first component is termed the *specific yield* (S_y) and is the amount of water produced by "de-watering" the aquifer void space as the water table falls within the aquifer. This term effectively determines the amount of water which is gained or lost under the district or some specific area due to the rising or falling water table. The values for well sorted sandy sediments[3], such as in the area of interest may range from $0.10 \leq S_y \leq 0.35$. The formula for calculating the volume of water released by dewatering is $V_w = A \cdot S_y \cdot \triangle h$ for a drop in water table of $\triangle h$. The aquifer thickness is not a term in this calculation.

Specific storage and Storativity. The second component is termed the *specific storage* (S_s) and is the much smaller amount of water produced by contraction of the dilated pore space and expansion of the water as the pressure drops within the aquifer. The values for loose- to well-packed silty or sandy sediments[4], such as in the area of interest may range from $0.00017 \leq S_s \leq$

[3]Fetter, C. W., 1994, Applied Hydrogeology, 3rd ed., Prentice - Hall, Inc., Table 4.4, p.91.

[4]Domenico, P.A. and Schwatrz, F.W., 1990, Physical and Chemical Hydrogeology, John Wiley, Inc, Table 4.1, p.111.

0.0032 ft^{-1}. This property is related to the in situ bulk compressibility of the aquifer media and the water itself. The compressibility of water is known and we can measure the compressibility of sediment samples, as SSS has done for RRBWSD on another project.

The formula for calculating the volume of water released from the aquifer by de-pressuring is $V_w = A \cdot H \cdot S_s \cdot \Delta h$ for a drop in head of Δh in an aquifer of thickness H. The product of aquifer thickness and specific storage in this equation is defined as *storativity*, $S = H \cdot S_s$, and it is obvious that if the thickness of an aquifer changes, then the value of S will change, even though the intrinsic property of the porous medium, i.e., the specific storage, remains constant. It should also be noted that if only a portion of the full thickness of an aquifer is relevant to a particular problem, then the appropriate value of S to be used in any calculation is the value for the interval of interest.

The specific storage term is also an essential term in the flow equations which describe transient, i.e. non steady- state, aquifer flow. The ratio of hydraulic conductivity to specific storage is defined as the hydraulic diffusivity and this ratio explicitly occurs in all non steady-state equations of flow. Therefore, while it is tempting to dismiss the need for an accurate value of S_s because it is negligible for the calculation of aquifer storage, it is important to obtain as good a value as is possible because it occurs directly in the flow equations along with conductivity. For example, the 20-fold difference between low and high values of S_s will make only a negligible difference in the calculated storage capacity of the aquifer, but will significantly alter the calculated results of the flow equations.

<u>Available storage capacity (SC).</u> The available storage capacity, which is not relevant to this particular scope of work, is defined as the volume of water that could be stored in the unsaturated zone above the water table within the boundaries of the District up to within a few feet of the ground surface. This working definition is usually used to calculate a change in aquifer storage due to a rise or fall of the water table over some time period, and is not an important or even relevant part of many types of aquifer modeling. In practice, the specific storage (S_s) is negligible compared to the specific yield (S_y) ($0.0005 < S_s/S_y < 0.00005$) so, unless an aquifer is very thick, we do not consider the specific storage in the calculation of storage capacity and just use the specific yield formula.

Layered- aquifer storage properties. For aquifers which are either heterogeneous or layered, we must determine the storage properties of each type of sediment within the aquifer and the proportions of each, and perhaps even the sequence in which they are successively filled and/or dewatered. For a layered aquifer, the volumetric storage capacity under an area (A) is defined as being equal to the volumetric integral: $SC = \int^A \int \phi(1 - S_r)dAdh$, which simplifies to a summation which looks like: $SC = A\Sigma(h_i \cdot Sy_i)$, that is, the total District area times the sum of products of the individual layer thicknesses and specific yields which, finally, is often more-conventionally written as: $SC = A \cdot H \cdot Sy_{eq}$, the product of total District area times total aquifer thickness times the "average" or equivalent specific yield. We must always remember that the correct values for determining an actual change in storage must be those values of $h_i \cdot Sy_i$ which represent the actual interval being filled or dewatered, and not the "full-aquifer" average value. Any modeling effort which simplifies the aquifer stratigraphy by reducing the number of layers must address the issue of determining the equivalent parameters of each layer model relative to the actual parameters of the actual stratigraphy.

The same additive property is true of storativity (S) for a sequence of layers in an aquifer. The value of storativity is a summation which looks like: $S = \Sigma(h_i \cdot Ss_i) = \Sigma(S_i)$, that is, the storativity of any depth interval is the sum of the individual- layer storativities for all layers within the interval. This additive property is important to consider when interpreting well tests which are not completed across the full layered- aquifer thickness.

Flow properties.

Water flows down-gradient in an aquifer from higher to lower potential. Groundwater flow may be horizontal or vertical or have components of both. The externally applied forces which cause water to move come primarily from gravity and secondarily from manmade actions. In other words, left to itself, the groundwater in aquifers and in basins "seeks its own level" and always prefers the path of least resistance. Water will stop moving when there is no change in potential along the pathway, otherwise water is moving in one of two type of conditions, either steady- state flow or transient flow. In steady state flow, water passing any location continues to flow in the same direction at the same flow rate and at the same head without any change over time. In transient flow, water passing any location will not be steady in direction, flow rate, or head because of dis-equilibrium somewhere in the system. Transient flow is non-Darcy flow and this is the condition of the aquifer in the project area most of the time.

The persistent re-equilibration of a groundwater system toward a no-flow condition takes time and often the cycle of recharge to- and recovery of- water from the system is faster than the ability of the groundwater system to either re-balance or even achieve a steady- state. As in most cases, the groundwater system is always dynamic and in a transient state, even if it appears to respond slowly and steadily by human perception.

The groundwater flow behaviors of interest include flow direction and flow rate. Flow direction may be visualized as an arrow pointing in the down- direction of the potential gradient, since water moves in the direction of the applied force. Flow direction may also be visualized as a hypothetical flowline that a single water molecule would follow under steady-state. A contour map of the water table or piezometric surface is a map of the groundwater potential in the aquifer, and the direction of flow at any location will be perpendicular to the contours and pointing in the direction of lower potential.

Flow rate can be described as the average flow speed of a water molecule at a specified place and time or as the "instantaneous" volumetric flux of a volume of water through a specified cross sectional area (W·H) of the aquifer over a short period of time. Apart from the externally applied driving forces and the physical dimensions of the aquifer, these measures of ground water flow depend only on the hydraulic conductivity and porosity of the porous medium.

Hydraulic conductivity. The *hydraulic conductivity (K)* is a measure of the ease with which water flows through a porous medium and is not quite a fundamental property. The fluid flow through a porous medium also depends on the density and viscosity of the fluid as well as what we call the *permeability (k)* of the porous medium. But for convenience hydrologists have combined the standard values for the properties of water with the measure of permeability and have defined this property as the hydraulic conductivity.

This term effectively determines the flow rate or volumetric flux of water through the aquifer under whatever potential gradient exists at the time and place of interest. The values for silty and sandy sediments, such as occur in the area of interest may range from $0.001 \leq K \leq$

300 ft/d, a range covering more than five orders of magnitude. The formula for calculating the steady- state volumetric flux of water (Q_w) in an aquifer is $Q_w = W \cdot H \cdot K \cdot G$ for a groundwater potential gradient $G = \Delta h / \Delta x$, through an aquifer cross sectional area of width W and thickness H. If the aquifer is not in steady- state, then the calculation represents only the "instantaneous" flow at that moment at that location under those conditions and the full equation of flow must instead be used to describe the transient flow behavior over time.

The steady- state flux equation applies to both horizontal and vertical groundwater flow with the condition that the values of K and G are the values of hydraulic conductivity and gradient in the direction of flow. Most aquifer materials, whether unconsolidated sediments or sedimentary rocks, are anisotropic and are commonly 5 - 20 times more permeable in flow directions parallel to the bedding planes than in flow directions perpendicular to the bedding planes. Thus, in order to quantify or model aquifer flow with both horizontal and vertical components, it is necessary to specify both the horizontal and vertical hydraulic conductivities of relevant aquifer materials.

Leakance. The *leakance (L')* is a property which determines the rate of downward vertical flow of water from a water table aquifer, through a permeable aquitard, and into an underlying semi-confined aquifer due to head differences across the aquitard. The value of L' is determined as the ratio of vertical hydraulic conductivity to the thickness of the aquitard, $L' = K_v'/h'$. (The prime (′) in the abbreviations symbolize that these are properties of the *aquitard* and not of the underlying aquifer). We refer to aquifers which show this type of recharge behavior as leaky aquifers and one flow equation which describes this type of flow behavior is the Hantush - Jacob equation, named after its authors.

The mathematics of leakage occurs in the flow equation in the form of what is referred to as the Hantush leakage factor (B) and B is related to known parameter values according to the formula $B = (T/L')^{1/2}$. In the project area, the high- permeability zones of the aquifer are sandy sediments and the low-permeability zones are silty sediments. These silty sediments are the aquitards which retard the vertical flow of water between the sandy layers of the aquifer. Based on our measurements and estimates of the relevant properties, we estimate that the value of B varies in the range of about $1800 \leq B \leq 6000$ and we have used a value of $B = 3200$ as our base case value.

Porosity. The *porosity* (ϕ) is the dimensionless ratio of the volume of void space in a unit volume of a porous medium. As a flow property, it determines the amount of intergranular flowpath within the porous medium that is available to the water. The formula for calculating the steady- state flow velocity of water (v_w) in an aquifer is $v_w = K \cdot G / \phi$. If the aquifer is not in steady- state, then the calculation represents only the "instantaneous" flow speed at that moment at that location under those conditions and the full equation of flow must instead be used to describe the transient flow behavior over time. For our modeling we have used an average porosity of 30%.

Layered- aquifer flow properties. For aquifers which are either heterogeneous or layered, we must determine the hydraulic conductivity and porosity of each type of sediment within the aquifer and the proportions of each. For a layered aquifer, the total average horizontal hydraulic conductivity of the full saturated aquifer thickness is defined as being equal to a summation which looks like: $K_{avg} = \Sigma(h_i \cdot K_i)/H$, that is, the sum of products of the individual layer thicknesses and hydraulic conductivities divided by the total aquifer thickness. Since the product of thickness and conductivity in this equation is defined as transmissivity (T), this is often more-conventionally written as: $T_{eq} = H \cdot K_{avg} = \Sigma(h_i \cdot K_i) = \Sigma(T_i)$, i.e., the equivalent aquifer transmissivity is the sum of the individual layer transmissivities.

The average conductivity and equivalent aquifer transmissivity refer to a hypothetical, homogenous aquifer which would deliver the same total volumetric flux as the specified layered aquifer. However, it must be remembered that the true flow behavior and volumetric fluxes are different in the individual layers of the actual aquifer than in the hypothetical equivalent- layer model and that the average or equivalent properties represent a mathematical fiction which is usable only in certain specific ways.

Transport properties.

Transport in this context refers to the motion of constituents which are dissolved and/or suspended in ground water, especially the movement of unregulated contaminant releases which propagate as slugs or plumes within the aquifer. The important transport processes are advection, dispersion, retardation, and attenuation which might be defined as follows. Advection is the physical transport of a constituent by the flow of water within a porous medium. Retardation includes all processes which cause a plume or constituents to move slower than the ground water. Dispersion includes all processes which re-distribute constituents away from the center of mass of a plume. Attenuation includes all processes which permanently remove constituent mass from a ground water plume.

These processes affect contaminant transport and plume behavior in specific ways. Mathematically, they may all be represented by terms in the transport equation which describes the location, speed, amount and distribution of contamination within the plume in space and time. Advection refers to groundwater flow which we have already discussed. Both retardation and attenuation may be thought of as properties related to the type of constituent rather than as properties of the aquifer. Dispersion is related to dispersivity which is strictly an aquifer property which can be measured with special types of well test or estimated from theoretical considerations. Since the treatment of transport is outside this project scope of work, we omit the discussion of these processes from this report. However, it is important to note that most contaminants travel at different flow speeds and different particle trajectories than the ground water and must be modeled in different ways.

Sources of data.

Sierra Scientific Services (SSS) used four sources of information for the aquifer properties within the area of interest (AOI) including:

1. SSS physical property data (S_y, S_s, ϕ, K, H, F_{sd}) measured on samples & logs from the AOI,
2. ID4 well test data (T & S) from wells in the AOI,
3. C.o.B. infiltration test data (Kv) for test ponds in the AOI,
4. KCWA water table elevation maps covering the AOI.

SSS carefully reviewed and chose not to use the data from two other sources including:

5. DWR aquifer model data (Sy, S, K & T) for the Kern Fan area,
6. KWBA and Pioneer Project pump test data from various reports (T & S).

SSS did not use the data from these two sources for several reasons, chief among them is that we obtained a minimum but sufficient amount of well- documented, actual measurements for all of the necessary parameters of interest from the first four sources. However, with all due respect, we also consider the data from both of these other two sources to be questionable and we recommend that ID4 carefully evaluate the data from these sources against their own technical and theoretical criteria before they use them in their own analyses. We offer some of our observations regarding the data from these two sources below, for purposes of clarification since we consider the selection (or rejection) of parameter values to be an important, documentable, exercise of judgment in a modeling program.

The DWR parameter data. In 1988, the California Department of Water Resources (DWR) purchased approximately 20,000 acres of land in Kern County for an aquifer storage and recovery (ASR) program. The area is now known as the Kern Water Bank. In the early 1990's, the DWR attempted to develop a computer model to simulate the aquifer behavior and evaluate various aspects of their project. The modeling effort concluded in early 1996 with the publication of a DWR memorandum which summarized the work. The memorandum included a discussion and summary of all the aquifer parameter values that the DWR used, and these parameter values have been referenced and used by some workers in the local water community.

In the process of parameterizing their computer model of the Kern Fan area, the DWR never actually measured a single value of any parameter in preparation for what became a massive modeling effort. The DWR assigned "textbook" values of specific yield obtained from the general literature (but not specific to the study area) to each of 55 different types of sediment. Then, after blundering through a simplistic and erroneous application of trend analysis in which all other parameter values were numerically correlated to the assigned values of specific yield, they proceeded to put these values into their computer model.

Apart from the parameter values, the DWR approach to developing the basic model appears to be sound and many of their model simplifications which were required in order to approximate the true physical aquifer behaviors within the constraints of the model show careful thought if not always the best choice. However, in our opinion, their treatment of aquifer parameters shows poor judgment perhaps stemming from an insufficient understanding of the physical properties and property interrelationships of porous media and geological materials.

DWR reported that the model results were unsatisfactory based on the initial values so they changed the parameter values around their control points to improve the outcome. Unfortunately, the DWR computer model never provided good results, which we attribute to incorrect parameter values, poor assumptions and poor choices of free parameters in the "calibration" tests. Since the initial parameter values were questionable on petrophysical and theoretical grounds and since the model results were unsatisfactory, we conclude that there is very little credibility in the representativeness of any of the DWR parameter values except to the extent that they fall within the ranges of published values for similar geological materials.

KWBA and Pioneer Project well test data. The operators of these two sites have conducted a number of pump tests on wells in these areas over the years, and the test data have been interpreted by other workers to provide estimates of the aquifer parameters T & S. With respect to the available literature, part of the issue with these well tests as a source of aquifer parameters is that the test operations and test data are only poorly documented in the available reference literature. But based on our review of the scant information in the literature, we can make the following observations.

The pump tests appear to have been designed and operated by engineers in order to determine well- function parameter values rather than aquifer parameter values. Many of these tests had multiple wells pumping simultaneously and most tests lasted for only a short duration, both of which make it difficult to determine aquifer properties. The available interpretations of aquifer transmissivity (T) appear to use the specific capacity (SC) approximation method rather than the standard Theis or Cooper - Jacob methods. Since the SC approximation method does not determine the aquifer storativity (S), the value of S must be guessed at in order to calculate the value of T. With possible values of S ranging over three orders of magnitude, this creates considerable uncertainty in the value of T, and none of the available reports discusses the specific value of S or the method of determination that was used.

If we speculate that the discharge/drawdown ratios (SC) are the only pump test data actually available from all of these tests, then the SC approximation method is all that is really available to anyone to estimate some T-values. However, we found that the determinations of more than two dozen values of T by the SC method were also poorly documented. Based on our review of the limited discussions, we conclude that there is evidence of a disregard for certain basic assumptions and limitations of the SC method which call the T-values into question. There was no discussion on the value of S which was assumed in the determinations of T nor any discussion of how the value was arrived at. There was no discussion on the value of well efficiency or whether or not it was included in the calculation, since the drawdown inside the pumping well must be corrected to fit the assumptions of the method. There was no discussion or mention of whether or not the T values were corrected for differences in pump test duration, effects of partial well penetration, length of completion interval, or differences in static head between wells.

When all of the T-values are looked at collectively, there is a surprisingly large variation within what is considered to be a "sweet spot" of aquifer geology. The reference reports attempt to explain this variation by citing differences in water table elevation at the different times of the tests. However, this explanation failed to explain why some wells had higher T-values when the water table was deep and lower T-values when the water table was shallow; failed to explain why none of the differences in T-values for different water table conditions were equal to the expected T-value of the dewatered interval, as it should be from theoretical considerations; and failed to consider the effects of different completion intervals in different test wells. Based on these observations and our questions regarding the undocumented and questionable use of the SC approximation method without the standard corrections, and further that the calculations are based on data from multi-well tests of short duration, we conclude that the more likely explanation for the difficulties with the T-values is that they are inaccurate and non- representative of the aquifer properties in the areas of the tests. We conclude that these data are of questionable value and have little credible use for aquifer modeling given the range of variability and the likelihood for a considerable range of error.

Parameter values.

Specific yield (S_y). Specific yield is a function of the porosity and grain surface area of porous media and is a property which varies over only a limited range of values for the few aquifer materials of interest. The source of specific yield values for this scope of work is the field work that SSS[5] completed for the Rosedale - Rio Bravo WSD and reported in 2003. RRB contracted SSS to drill coreholes, collect sediment samples and obtain laboratory analyses for specific yield and a set of other useful physical properties. One suite of samples which came from the RRB recharge pond area which is part of the ID4/ KT/ RRB project area.

Based on the RRB study, the average specific yield of the sandy and silty sediments in the area of interest are 33% and 8.6%, respectively. Based on the relative fractions of each in the upper aquifer, the average specific yield of the interval is about 21%.

---

[5]Crewdson, Robert A., 20 January, 2003, Determination of Aquifer Storage Capacity for the Rosedale - Rio Bravo Water Storage district, Bakersfield, California., Sierra Scientific Services, Bakersfield, Ca.

Specific storage (S_s). Specific storage is a function of the porosity and bulk compressibility of porous media and is a property which varies over only a limited range of values for the packed, unconsolidated sandy and silty media of interest. The source of specific storage values for this scope of work are also from the 2003 field work that SSS completed for the Rosedale - Rio Bravo WSD. Based on compressive stress tests on samples of poorly sorted sand and silty sand, the bulk compressibilities of these samples range from 4.5 - 7.9 x 10^{-8} m^2/N from which we have derived the values for the dense, compacted equivalents of these sediments as 1 - 1.8 x 10^{-8} m^2/N which are in the expected range of compressibilities for dense sands. From these values we have calculated the corresponding values of S_s ranging from 0.000030 to 0.000053 ft^{-1} and averaging 0.000041 ft^{-1}. This range of S_s values is entirely consistent with the range of published values expected for dense sands and silts.

Porosity (ϕ). The source of porosity values for this scope of work are also from the 2003 field work that SSS completed for the Rosedale - Rio Bravo WSD. Based on those samples, the measured average porosity of well sorted sandy sediments is 37% and the measured average porosity of the silty sediments is 34% and give a weighted average porosity for the aquifer media of 30% for this project.

Transmissivity (T) and Storativity (S). The source of T and S values for this scope of work are based on the ID4 well test of December, 2002 which has been summarized in a 2003 KDSA impact report and a supplemental well test report[6] (see Appendix 5). Based on our review of the supplemental report, we disagreed with the interpretation of T & S values because of a failure to select values of Q and r which met the necessary Cooper - Jacob assumptions and validity conditions for the method. We also disagreed with the entire distance - drawdown interpretation presented in the same report because of an incorrect application of the Cooper - Jacob (single-well) method to a cluster of several pumping wells. As a result, we have chosen not to use the reported values of T & S, in favor of our own reinterpretation of the data.

Based on our re-analysis (Appendix 4) of the Cooper - Jacob time - drawdown data, the correctly determined value of transmissivity is T = 20,000 ft^2/d and the correctly determined

[6]Schmidt, Kenneth, D., January, 2003, Groundwater Conditions and Potential Impacts of Pumping for the ID-4 Kern Parkway and Rosedale - Rio Bravo WSD Projects, Kenneth D. Schmidt & Associates, Fresno, CA, and Supplement to the [January Report], by the same source, dated February 28, 2003.

value of storativity is $S = 0.00056$ for the slotted intervals of the tested wells. As we report in Appendix 4, we are unable to determine any other values of T or S from any of the other data from this multi- well, 20-day test. Apart from the correctness of the calculations, we cannot determine the accuracy or representativeness of these T & S values because of a lack of corroborating data and because of known factors associated with the operation of this test which may have affected the data.

The calculated value of T is reasonable in our opinion based on experience. But because there is a body of other data, even though of questionable value in our opinion, which reports a wide range of possible and particularly much higher values of T, we have treated T as one of our free parameters which we vary in our modeling.

In our opinion, the calculated value of S seems to be too low for an aquifer which is at least 500 ft thick and for test- well completion intervals which cover a significant fraction of the full aquifer thickness. We point out that $S = H \cdot S_s$, and for S_s in the range of 0.000041 and H in the range of 300 - 500 ft, the storativity should be in the range of $0.01 < S < 0.02$. The very small measured value of S from the well test would suggest that either the effective aquifer thickness is actually quite small or that the validity of the entire test should be questioned. However, since part of the scope of work includes an evaluation of long- term drawdown impacts and since the steady- state aquifer behavior does not depend on the value of S, we take some consolation against our concern for the value of this parameter. However, the value of S is a factor in transient aquifer behavior, particularly in the size and expansion of the capture zone about a well so, to the extent that the value of S may be in error, this part of the modeled transient behavior may be in error as well.

Hydraulic conductivity (K). For the following two assumptions that 1. the fraction of sand (F_{sd}) by thickness in the aquifer is at least 20%, and 2. the sand is at least 100 times more permeable that the silt, we can demonstrate that the horizontal hydraulic conductivity (K) of the aquifer equals $K = T/(H \cdot F_{sd})$ with an error of less than 5%. Based on the geological cross sections presented in the first KDSA report, we estimate that the test wells were completed across an estimated 250 ft net sand interval in the local area of the well test, so that the hydraulic conductivity of the sandy strata must be about $K_{sd} = 80$ ft/d. However, we note that if the effective net sand thickness is much smaller in the region of this well test, especially if the flowpath between the pumped well and the observation well is stratigraphically limited, then the value of K might be very much higher for this value of T.

Aquitard leakage factor. The mathematics of leakage occurs in the flow equation in the form of what is referred to as the Hantush leakage factor (B) and B is related to known parameter values according to the formula $B = (T/L')^{½}$. In the project area, the high-permeability zones of the aquifer are sandy sediments and the low-permeability zones are silty sediments. These silty sediments are the aquitards which retard the vertical flow of water between the sandy layers of the aquifer. Based on our measurements and estimates of the relevant properties, we estimate that the value of B varies in the range of about $1800 \leq B \leq 6000$ and we have used a value of B = 3200 as our base case value.

Based on the local geology as interpreted from available E-logs, the shallow water table aquifer is separated from the underlying semi- confined aquifer by a sequence of thin, laterally-discontinuous, localized silty facies rather than a single, laterally- continuous aquitard. Since the silty strata are interbedded with more permeable sandy strata, the downward vertical flow of recharge to the aquifer is a non-uniform flux of slower vertical flow through the retarding silty strata and faster vertical flow through the sandy strata. If the aquitard is to be viewed or treated as a single layer, then it might look like a slice of swiss cheese, i.e., what we might call a "leaky aquitard". The overall leakiness of the aquitard is neither that of the silts alone nor of the sands alone, but a composite intermediate value of both which cannot be determined from the limited E-log data in the project area.

Appendix 4.

Cooper - Jacob analysis of the ID4 December, 2002 Well Test.

Appendix 4.

Cooper - Jacob analysis of the ID4 December, 2002 Well Test.

The Kern County Water Agency conducted a pumping test of several of their wells in the ID4 area of interest (see Appendix 5) in December, 2002. Kenneth D. Schmidt & Associates interpreted the well test data and presented the results in a pair of reports dated in early 2003.

To summarize, ID4 owns seven wells, six of which (Nos. 11, 10, 3, 9, 8, & 1 from SW to NE) have been installed along a line bearing N65E, parallel to the Kern River channel over a distance of about 10,800 ft and centered just NE of the southeast corner of section 36, T29S/R26E. On December 10, 2002, ID4 turned on all of these wells and let them flow at constant rates for 20 days. The flow rates in the six wells ranged from 5.6 - 10 .0 cfs and ID4 extracted a total of 1890 af from the aquifer at an average rate of 94.5 af/d over the 20-day duration of the test. ID4 monitored the flow rates and drawdowns in each pumping well over time and monitored the drawdowns in selected other non-pumping wells in the immediate area.

<u>Cooper - Jacob time - drawdown analysis.</u> ID4 recorded the drawdown in observation well ID4-12 several times over the first two days of pumping. This observation well is approximately equidistant from the two closest pumping wells (3 & 10) and much more distant from the other pumping wells. The KDSA Supplemental Report (SR) presented these drawdown data for well ID4-12 on a log-linear Cooper - Jacob time - drawdown plot and calculated values of T & S from the data. The standard procedure is to calculate T from the slope of the straight line plot of the data, and then calculate S from the x-intercept and the value of T. Although the method and calculation are not presented in the SR, we assume that *the KDSA calculations* were approximately as follows for Q = 23,850 gpm, r = 2240 ft, slope = 13.2 ft/decade, and intercept = 0.03 day.

$$T = 2.3Q/(4\pi \text{slope}) = \text{for } Q = (2.3)\cdot(23{,}850\cdot 1440)/((12.56)\cdot(13.2)) \approx 476{,}200 \text{ gpd/ft}$$

and

$$S = 2.25T(\text{intercept})/r^2 = (2.25)\cdot(476{,}200)(0.03)/((7.48)\cdot(2240)^2) \approx 0.00086$$

In our opinion, the values of Q and r which were used in these calculations are incorrect. The value of Q = 23,850 gpm is the total combined pumping rate of all six wells. But based on Cooper - Jacob validity criteria, only wells 3 & 10 are close enough to the observation well to have caused measurable drawdowns after 2 days of pumping, and the other wells are too far away. Therefore, if the effects of pumping at wells 1, 8, 9, & 11 have not yet reached the observation well, then the flow rates of these wells should not be included in the calculation. The combined Q of wells 3 & 10 is about 7,500 gpm so we conclude that this is the appropriate value of Q for this test, subject to other validity criteria.

With respect to the value of "r", The value of r = 2,240 ft is the distance from the observation well to the centroid of all six pumping wells (SR Table 3). But based on Cooper - Jacob validity criteria, the Cooper - Jacob calculation method only applies to the observed drawdown caused by one of two cases, either 1. a single pumping well at a distance r from an observation well, or 2. multiple wells ALL of which must be at the same distance r from the observation well AND all of which must start pumping at the same time. This method does not apply to, i.e., does not give the correct results from, any other set of conditions and does not apply to the "centroid" of a multi-well cluster of pumping wells.

We note, however, that the placement of observation well 12 with respect to pumping wells 3 & 10, approximately meets the second of the two C-J validity conditions with a value of r = 1,540 ft to both wells as determined from the KDSA map (Schmidt, 2003b, Figure 1).

Therefore, based on our re-analysis using Q = 7,500 gpm (1,444,000 cf/d) and r = 1,540 ft, and the SR values of slope and intercept which we agree with, then we determined (after a units conversion) the local value of transmissivity to be T = 20,000 ft^2/d and the value of storativity to be S = 0.00056 for the slotted intervals of the tested wells, as follows:

$$T = 2.3Q/(4\pi \text{slope}) = (2.3)\cdot(1{,}444{,}000)/((12.56)\cdot(13.2)) = 20{,}000 \text{ ft}^2/\text{d},$$

and

$$S = 2.25T(\text{intercept})/r^2 = (2.25)\cdot(20{,}000)/(1540)^2 = 0.00056.$$

<u>Cooper - Jacob distance - drawdown analysis.</u> The SR presented a distance - drawdown plot of the drawdowns in non-pumping wells after a pumping duration of 10 days. In our opinion, the

observed scatter in the plotted data (Schmidt, 2003b, Figure 3) is in obvious contradiction to the expected Cooper - Jacob straight line, and is sufficient to cause us to question the data. It appears that the drawdowns from a number of non-pumping wells have been plotted against the radial distance from the geographic centroid of the six pumping wells. But based on Cooper - Jacob validity criteria, the Cooper - Jacob distance - drawdown calculation method only applies to the observed drawdowns at various distances at a single specified time caused by a single pumping well. This method does not apply to, i.e., does not give the correct results from, any other set of conditions and does not apply to the "centroid" of a multi-well cluster of pumping wells, as should be obvious from geometrical considerations alone. Therefore, we have rejected the results of that analysis based on incorrect methodology. Unfortunately, there is no method of interpretation for to determine T & S from a cluster of multiple pumping wells and no way to isolate the component of drawdown in an observation well that is due to just a single pumping well.

Appendix 5.

Excerpts from the KDSA, 2003b, Supplemental Report regarding the ID4, December, 2002, Pump Test.

KENNETH D. SCHMIDT AND ASSOCIATES
GROUNDWATER QUALITY CONSULTANTS
3701 PEGASUS DRIVE, SUITE 112
BAKERSFIELD, CALIFORNIA 93308
TELEPHONE (661) 392-1630

February 28, 2003

Mr. Martin Varga
District Engineer, ID-4
Kern County Water Agency
P. O. Box 58
Bakersfield, CA 93302-0058

Re: Allen Road Well Field
December 2002 Pump Test

Dear Martin:

Following are the results of my review of the December 2002 pump test on the six ID-4 Wells. Wells No. 1, 3, 8, 9, 10, and 11 were pumped for the test. Pumping of the wells began between 8:23 and 9:29 am on December 10, 2002. Pumping of Well No. 10 stopped at 7:20 pm on December 20, 2002, due to a pump malfunction. Pumping of the remaining wells continued until between 10:35 and 11:35 am on December 30, 2002. Figure 1 shows the locations of these wells and observation wells that were used for the test. A network was developed of both shallow and deep observation wells. There were a total of 13 deep observation wells and three "shallow" observation wells.

<u>Pumpage</u>

Total pumpage and average pumping rates were determined for the first day, next three days, first ten days (through December 20, 2002), and for the entire duration of pumping. Table 1 summarizes pumpage from each of the pumped wells for these periods, based on totalizer readings from the flowmeters on each well. A total of 34,716,000 gallons was pumped during the first day, 101,815,000 gallons during the next three days, 384,900,000 gallons during the first 10 days, and 615,943,000 gallons during the entire 20-day period. The average pumping rates were 24,110 gpm for the first day, 23,570 gpm for the next three days, 23,260 gpm for the first 10 days, and 21,390 gpm for the entire test. Because of the combined well decreased pumping rate after December 20, drawdowns in the pumped wells and observation wells were primarily examined during the first ten days of the pump test.

Drawdowns

Pumped Wells

Table 2 summarizes drawdowns in the pumped wells that were determined for the test. The centroid of pumping was located about midway between Wells No. 3 and 9. Static water levels in these wells ranged from 127.8 to 131.9 feet, and were deepest to the northeast. Drawdowns in these wells after 10 days of pumping ranged from 31.7 to 55.7 feet. The largest drawdowns were in Wells No. 1 and 3, even though these wells had the lowest pumping rates. Drawdowns after 20 days of pumping ranged for 31.5 to 53.3 feet.

Observation Wells

The primary observation well for the test was ID4 Well No. 12, which is in the vicinity of the pumped wells, and was not pumped during the test. Table 3 shows drawdowns in the deep and shallow observation wells that were measured during the test. The distances of these wells from the centroid of pumping are also shown.

Although three observation wells were initially chacterized as "shallow", only one of these (2-inch galvanized) is believed to tap only the shallowest deposits (Layer 1). The water-level in this well rose during most of the pumping period, and was directly influenced by recharge from streamflow in the Kern River. Well 35A4, located near the RRBWSD office, is perforated from 310 to 410 feet in depth and is actually an intermediate zone (Layer 2) well. The water level in the shallow nearby well (Shop Well) couldn't feasibly be measured during the test. The depth of the Hay Barn Well is not known, but the depth to water in this well is also indicative of an intermediate zone well. Two other shallow monitor wells were planned to be measured during the test, but they were dry. Because of these factors, the pump test did not provide useful information on the influence of pumping the ID4 Allen Road wells on shallow groundwater in the vicinity. However, substantial information was obtained on aquifer characteristics for the composite pumped interval.

Figure 2 is a drawdown plot for Well No. 12. The measurements for the first two days of pumping indicated a transmissivity of 476,000 gpd per foot and storage coefficient of 0.0008. Water-level measurements in this well after six days of pumping the other wells indicated no further drawdown. This was due to recharge from streamflow in the Kern River, which began between December 12 and

16, 2002, and reached the ID4 Well No. 10 vicinity by December 19.

Drawdowns in deep wells (most are actually composite wells) ranged from 1.3 to 26.3 feet after ten days of pumping the ID4 wells. Figure 3 shows drawdowns in these wells after ten days of pumping plotted against the logarithm of distance from the centroid of pumping. The overall trend is excellent, and a transmissivity of 395,000 gpd per foot and storage coefficient of 0.01 were determined. This transmissivity value is in excellent agreement with the average value of 409,000 gpd per foot used in our January 2003 report. The increase in apparent storage coefficient (ten days versus two days) is as expected. Afer several months of pumping the ID4 Allen Road Wells, the storage coefficient is expected to be in the range of 0.05 to 0.10.

Recovery

Recovery measurements indicated almost full recovery within the first day after pumping stopped. Unfortunately, frequent water-level measurements were not made during the first day of recovery. Therefore, values for aquifer characteristics could not be determined from the recovery measurements.

Please call me if you have any questions.

Sincerely Yours,

Kenneth D. Schmidt

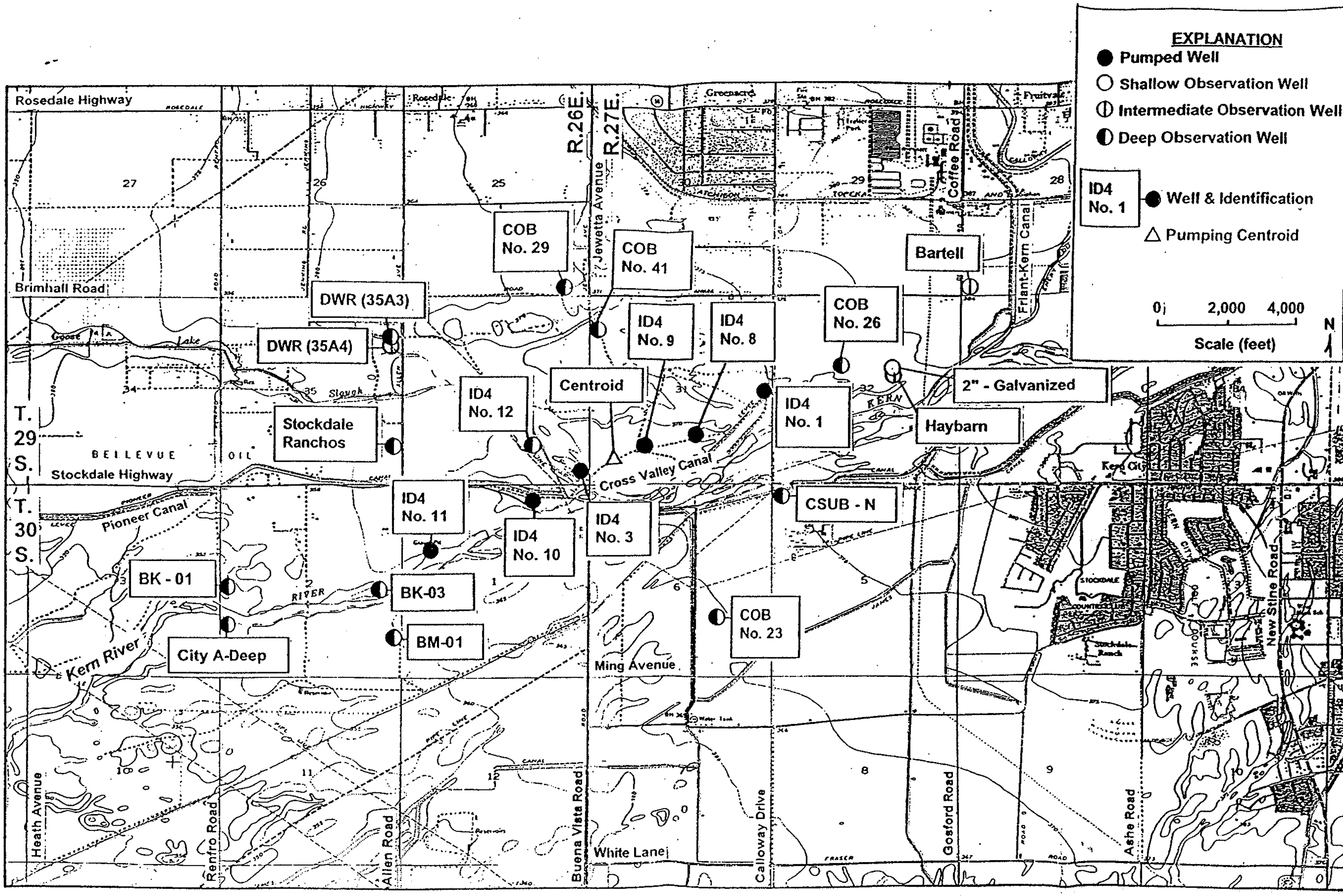


FIGURE 1 - LOCATION OF PUMPED WELLS AND OBSERVATION WELLS

TABLE 1 - PUMPAGE AND PUMPING RATES
FOR THE PUMP TEST

	First Day		Next Three Days		First Ten Days		Entire Test	
Well	1,000g	Ave gpm	1,000g	Ave gpm	1,000g	Ave gpm	1,000g	Ave gpm
1	3,991.7	2,770	11,743.7	2,730	39,209.6	2,730	79,031.9	2,730
3	4,359.9	3,020	12,760.3	2,940	41,969.6	2,920	86,627.5	2,990
8	6,578.9	4,570	19,319.7	4,470	63,407.4	4,410	127,749.9	4,410
9	6,585.5	4,600	19,182.9	4,440	62,863.2	4,370	126,863.6	4,390
10	6,578.9	4,550	19,287.1	4,490	63,446.5	4.410	66,216.2	2,300
11	6,621.3	4,610	19,521.7	4,510	64,003.7	4,450	129,454.1	4,480
Total	34,716.2	-	101,815.4	-	334,900.0	-	615,943.2	-

TABLE 2 - DRAWDOWNS IN PUMPED WELLS

Well No.	Static Level (ft)	December 20, 2002		December 30, 2002	
		Pumping Level (ft)	Drawdown (ft)	Pumping Level (ft)	Drawdown (ft)
01	131.6	187.3	55.7	184.9	53.3
03	125.8	180.8	55.0	176.7	50.9
08	132.1	179.2	47.1	177.7	45.6
09	131.9	163.6	31.7	163.4	31.5
10	129.1	178.0	48.9	Pump Off	-
11	127.8	179.1	51.3	174.2	46.4

TABLE 3 - DRAWDOWNS FOR OBSERVATION WELLS

				December 20, 2002		December 30, 2002	
Zone	Well No.	Distance to Centroid(ft)	Static Level(ft)	Depth to Water(ft)	Drawdown(ft)	Depth to Water(ft)	Drawdown(ft
Shallow	2" Galvanized	8,320	87.6	86.0	-1.6	84.2	-3.4
Intermediate	DWR-35A4	17,150	122.5	131.9	8.5	129.8	7.3
	Bartell	11,250	118.0	132.5	14.5	132.0	14.0
	Hay Barn	8,320	114.3	117.1	2.8	116.5	2.2
Deep	ID4-12	2,240	124.4	150.7	26.3	149.1	24.7*
	COB-23	5,330	145.1	159.2	14.1	156.1	11.0
	COB-26	6,990	134.2	154.1	19.9	153.3	19.1
	COB-29	4,960	139.1	150.0	10.9	148.6	9.5*
	BM-01	7,870	127.4	131.1	3.7	131.3	3.9
	BK-01	11,680	142.1	146.0	3.9	146.7	4.6
	BK-03	7,570	127.8	134.2	6.4	133.1	5.3
	CSUB-N	5,120	133.5	151.8	18.3	150.8	17.3*
	COB 41	3,680	136.7	145.8	9.1	143.1	6.4
	Stockdale Ranchos	6,190	125.0	130.5	5.5	130.0	5.0
	DWR-35A3	7,150	124.5	132.4	7.9	131.3	6.8
	City A-Deep	11,970	124.3	125.6	1.3	126.2	1.9

*Pumping levels and drawdowns were for December 27.

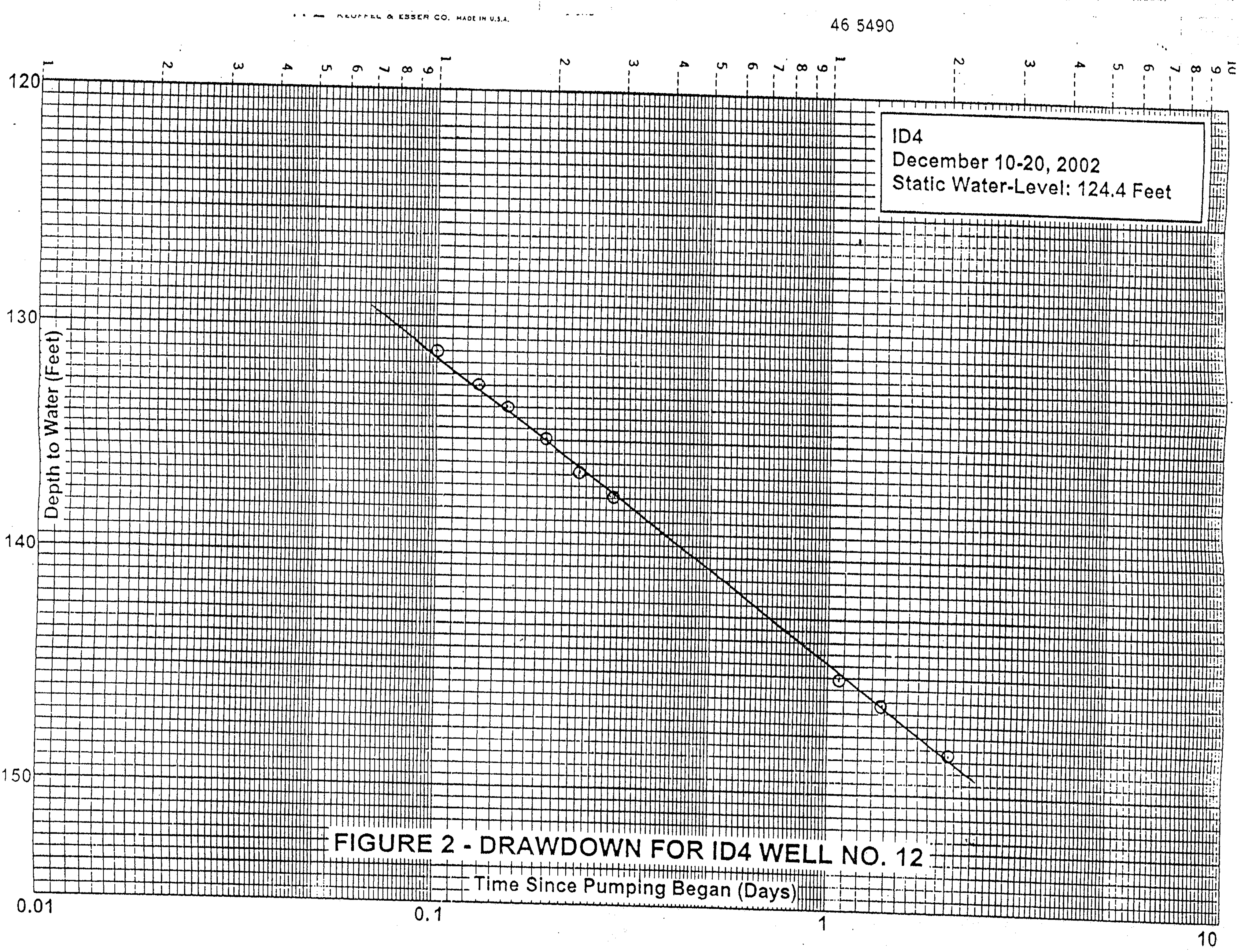


FIGURE 2 - DRAWDOWN FOR ID4 WELL NO. 12

46 5490

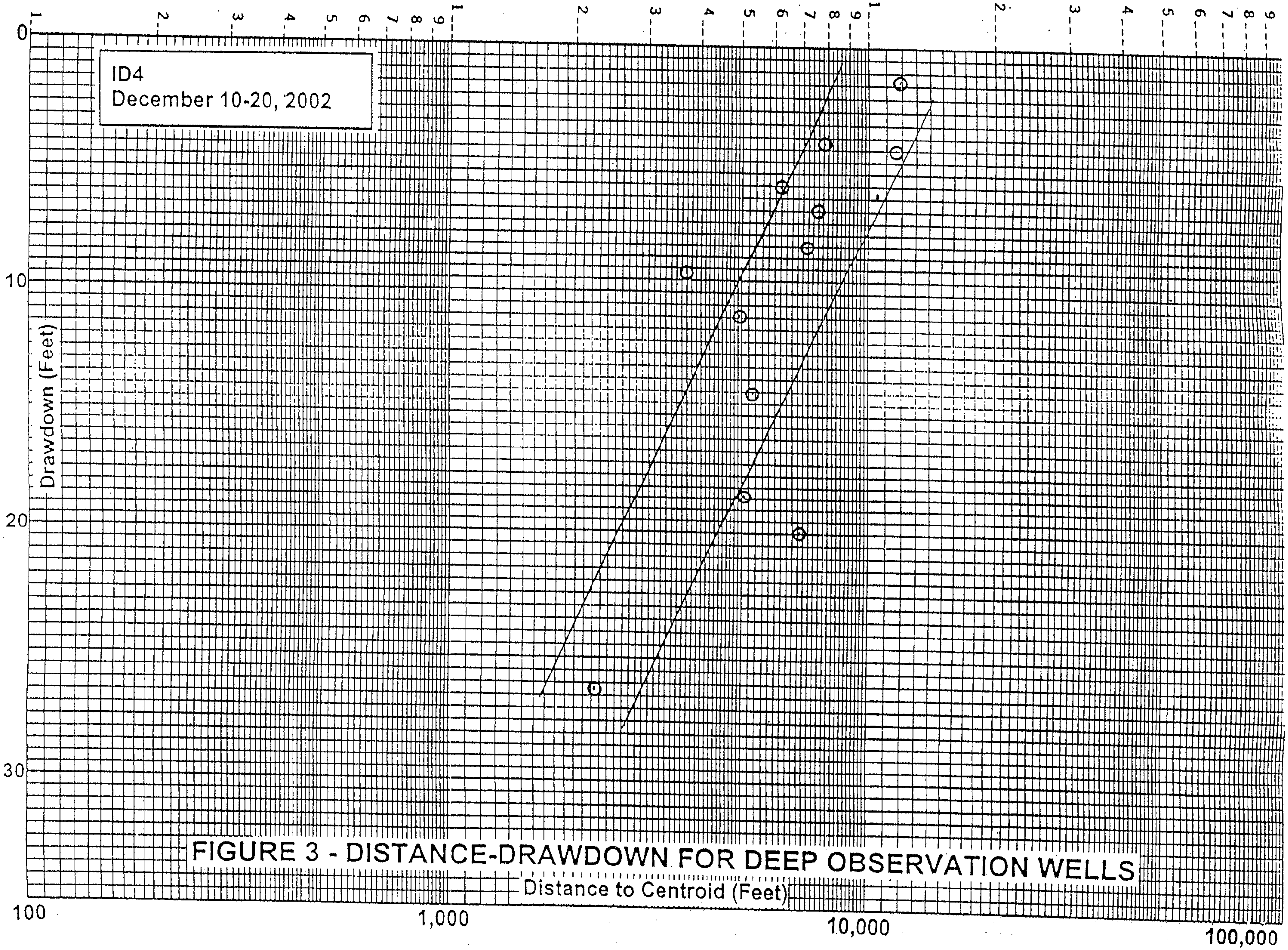


FIGURE 3 - DISTANCE-DRAWDOWN FOR DEEP OBSERVATION WELLS

Appendix 6.

Catalog of Drawdown Analyses for non- Base Case Conditions.

Appendix 6.

Catalog of Drawdown Analyses for non- Base Case Conditions.

SSS has evaluated several sets of non- base case conditions to illustrate the calculated drawdowns for purposes of comparison and evaluation. We have listed the sets of conditions below and have discussed each in subsequent sections. The best way to compare variations is to look at the maps to observe the changes in drawdown at locations of interest with respect to the changes in the free parameters and remember that the parameter changes are intended to reflect hypothetical changes in the real aquifer properties which affect the groundwater behavior.

We have compiled a set of introductory maps (Set 0) showing the well locations and the various groundwater gradient scenarios in the absence of project pumping. We have compiled the data sets into a group of drawdown analyses (Sets 1 - 6), a group of particle trajectory and capture analyses (Sets 7 - 8), and a group of other analyses (Sets 9-10). The difference is only in emphasis because the same basic map output is provided for every model calculation. From the drawdown analyses, we have tabulated the observed drawdowns at perimeter distances of 1000, 3000, and 5000 ft from the array of seven pumping wells which are presented in Figure 14. For the particle trajectory and capture analyses, we have tabulated (Figure 15) the capture reach (the equivalent of capture radius for a multi-well array) of the well array for pumping times of 1, 2, 5, 10, 20, and 30 years.

We present a list of the complete set of drawdown analyses in the catalog on the following page. We then present discussions of every set of maps in the remainder of this Appendix along with the catalog of maps.

List of Drawdown Analyses.

Set 1. Base case and limiting cases.

- Leaky aquifer, t = 300, 770 days (equiv. to 3 project years)
- Confined aquifer, t = 300, 770 days
- Unconfined aquifer, t = 300, 770 days

Set 2. Variation of base case drawdown with leakage rate (B) for constant t = 300d.

- B = 1800, 2200, 2600, 3200, 4600, 6000, 10000 ft

Set 3. Variation of base case drawdown during pumping at $Q = Q_{full}$.

- t = 10, 30, 100, 300, 770 days

Set 4. Modified base case by varying pump duration (t) at constant $Q = Q_{full}$.

- t/t_{max} = 100%, 50%, 25% t = 300, 150, 75 days

Set 5. Modified base case by varying flow rate (Q) for constant pumping time t = 300d.

- Q/Q_{max} = 100%, 50%, 25% Q = 90, 45, 22.5 cfs.

Set 6. Modified base case by pumping only selected wells for t = 300d.

- A1 - A5 only; ID4 1&2 only; A1 & ID4-2 only.

List of Particle Trajectory and Capture Analyses.

Set 7. Base case superimposed on a natural GW gradient (t = 300, 720d).

- G = -0.002; bearing 270° & 315° (rt azim. from North).

Set 8. Continuous pumping superimposed on a natural GW gradient (t = 1, 3, 5, 10, 20 yr).

- G = -0.002; bearing 270° & 315° (rt azim. from North).
- G = -0.001; bearing 300° and 0.7 ft/d recharge in Kern River channel.

List of Other Analyses.

Set 9. Base case by superposition method compared to centroid method.

Set 10. Three hypothetical pond infiltration scenarios.

Appendix 6.

Discussion
SET 00.

Summary of Introductory Maps; Set 0.

Set 0. Location Map and Ground water gradients.

This set of introductory maps includes five basic features. The first map is a location map showing several well locations and three reference- location markers positioned at local street intersections. The second and third maps show two hypothetical ground water gradient scenarios assuming an aquifer of infinite lateral extent. The two gradients are northwesterly at 0.002 and westerly at 0.002, respectively. These two ground water gradients are intended to represent the average gradients associated with the wet and dry periods, respectively, of the climatic wet/dry cycle. The fourth and fifth maps show a hypothetical west- northwesterly ground water gradient which includes the same upgradient recharge boundary at the Kern River channel, but have particle trajectories drawn on each for pumping periods of 3 years and ten years, respectively, for illustrative purposes without any pumping wells.

Appendix 6.

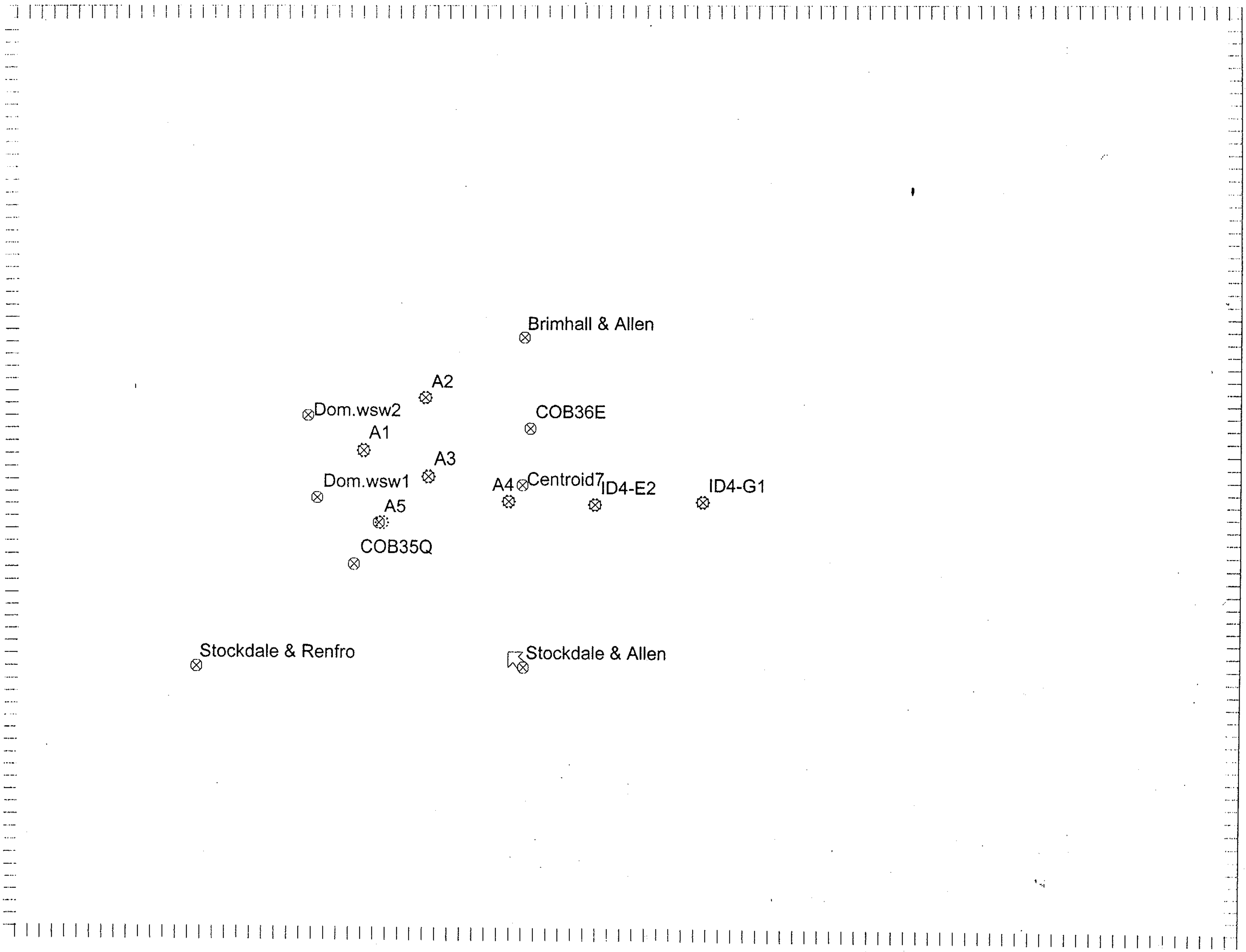
SET 00.

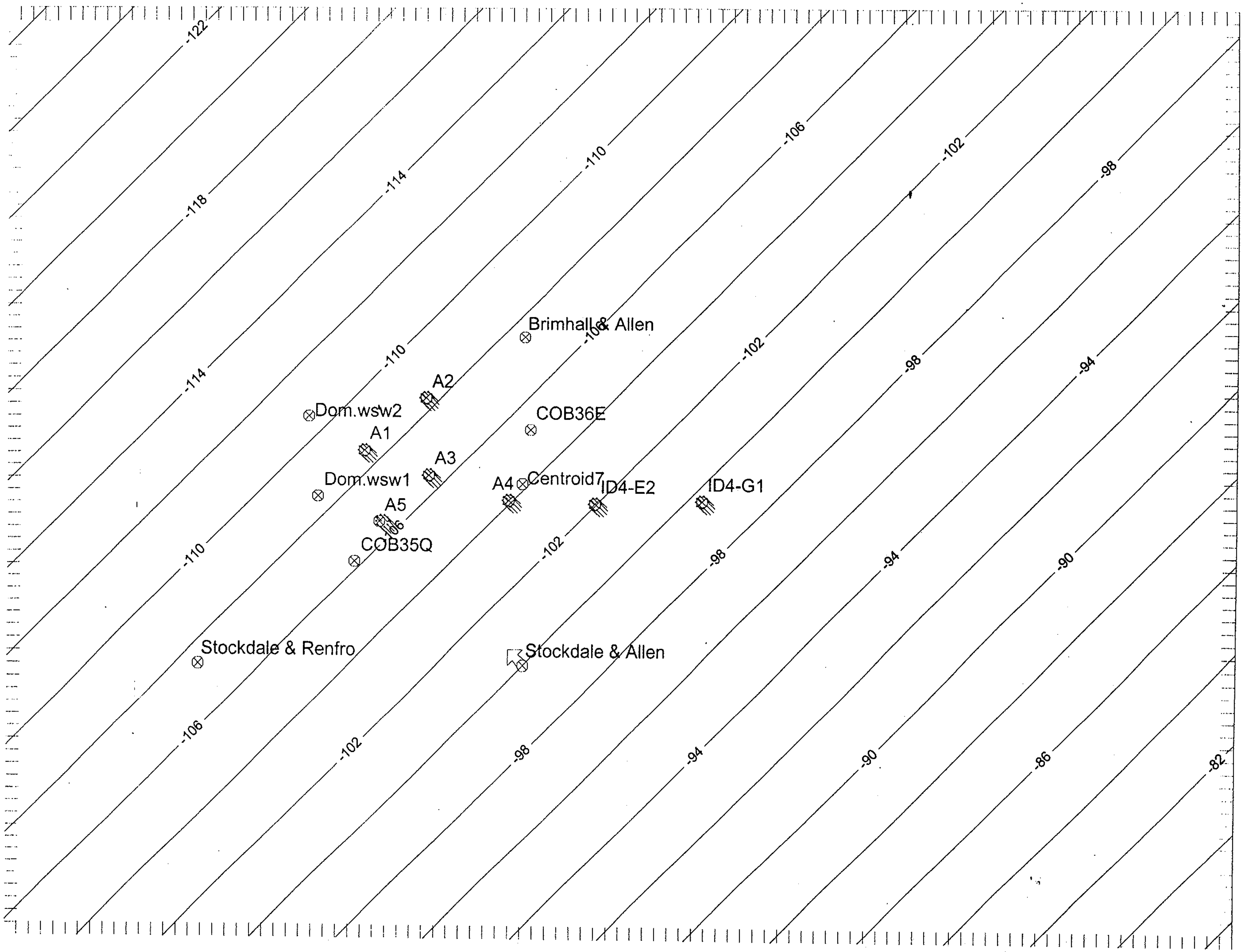
ID4 / KT / RRB Well Field Model Parameters.

Set No. 0.

Note: only CHANGES are tabulated between Runs, from left to right.
Note: once made, CHANGES persist until modifed again.

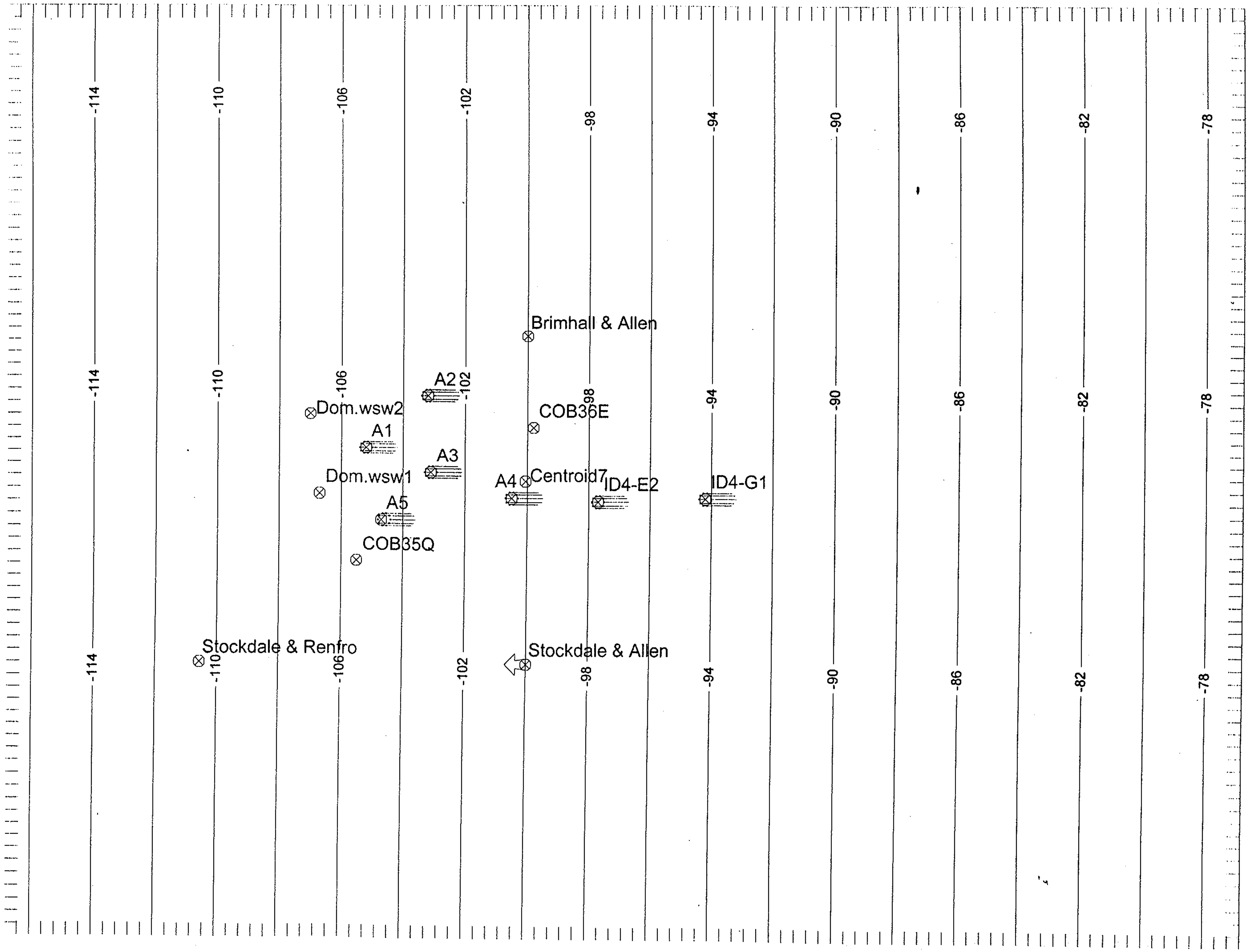
		base case												
model		leaky	loc.map	GW grad	GW grad	GW grad	GW grad							
calc		transient												
Run No.		ref.only	001	002	003	004	005							
H.Cond.	K	80												
Btm	B	-450												
Top	T	-200												
Ref Hd	R	0		-100										
Grad	G	0												
Rchg	Rch	0												
Por	P	0.3												
Stor	S	5.60E-04												
Lkg	L	3200												
pump-t	t	300	0											
Ref Hd	Ref	0		-100										
Grad	G	0		0.002		0.001								
Azim	A	135			180	150								
Ref-X	X	0												
Ref-Y	Y	0												
Const Hd	CH	no												
trace-t	tt	300	0	300	770		3653							
Flow Rate	Q	full	0											
Wells	W	all	none											
River Rch	Q	0				-210								


Brimhall & Allen
A2
Dom.wsw2
COB36E
A1
A3
Dom.wsw1
A4
Centroid7
ID4-E2
ID4-G1
A5
COB35Q
Stockdale & Renfro
Stockdale & Allen



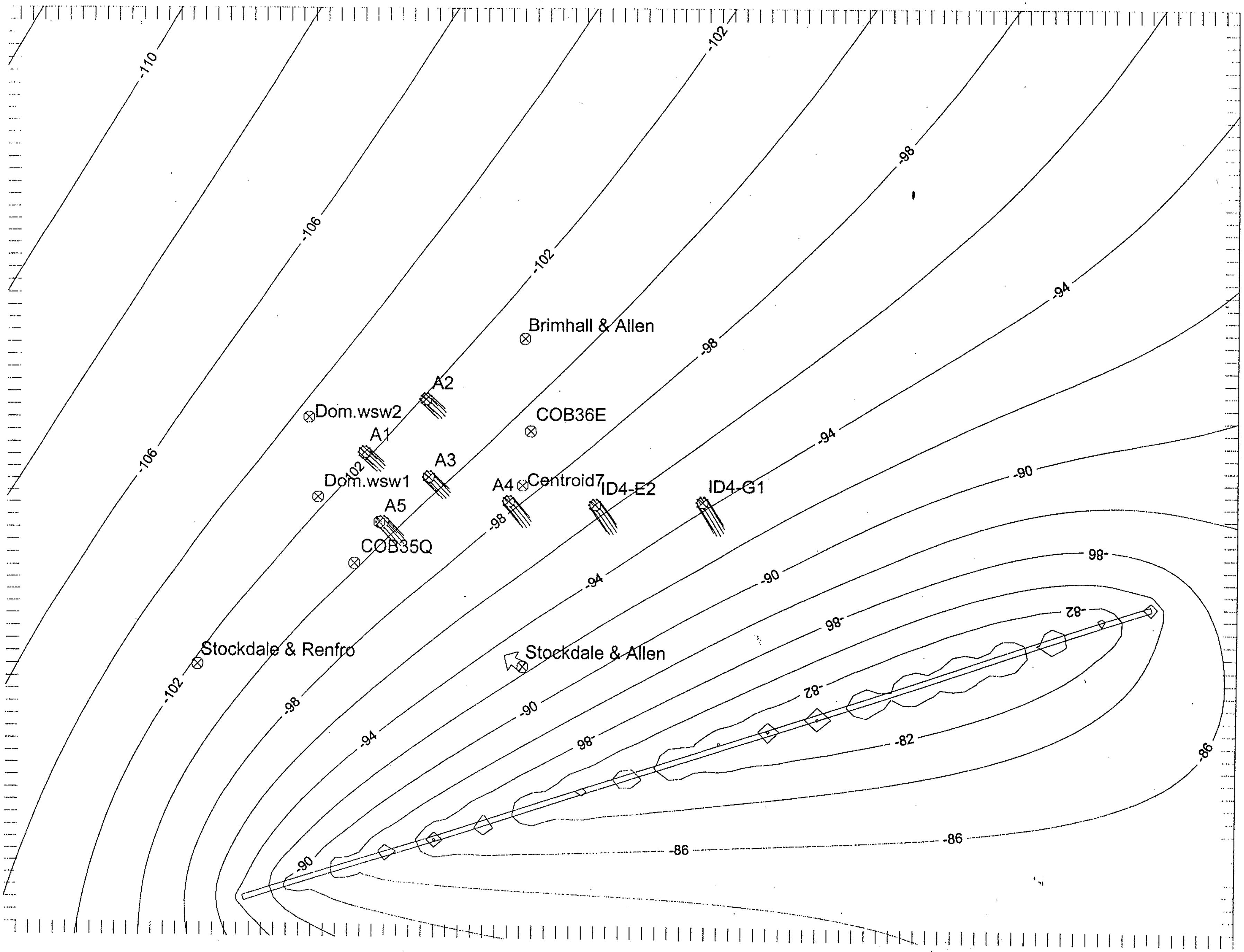
-122
-118
-114
-110
-106
-102
-98
-94
-90
-86
-82
Brimhall & Allen
COB36E
Dom.wsw2
A2
A1
A3
Dom.wsw1
A4
Centroid7
ID4-E2
ID4-G1
A5
COB35Q
Stockdale & Renfro
Stockdale & Allen

302

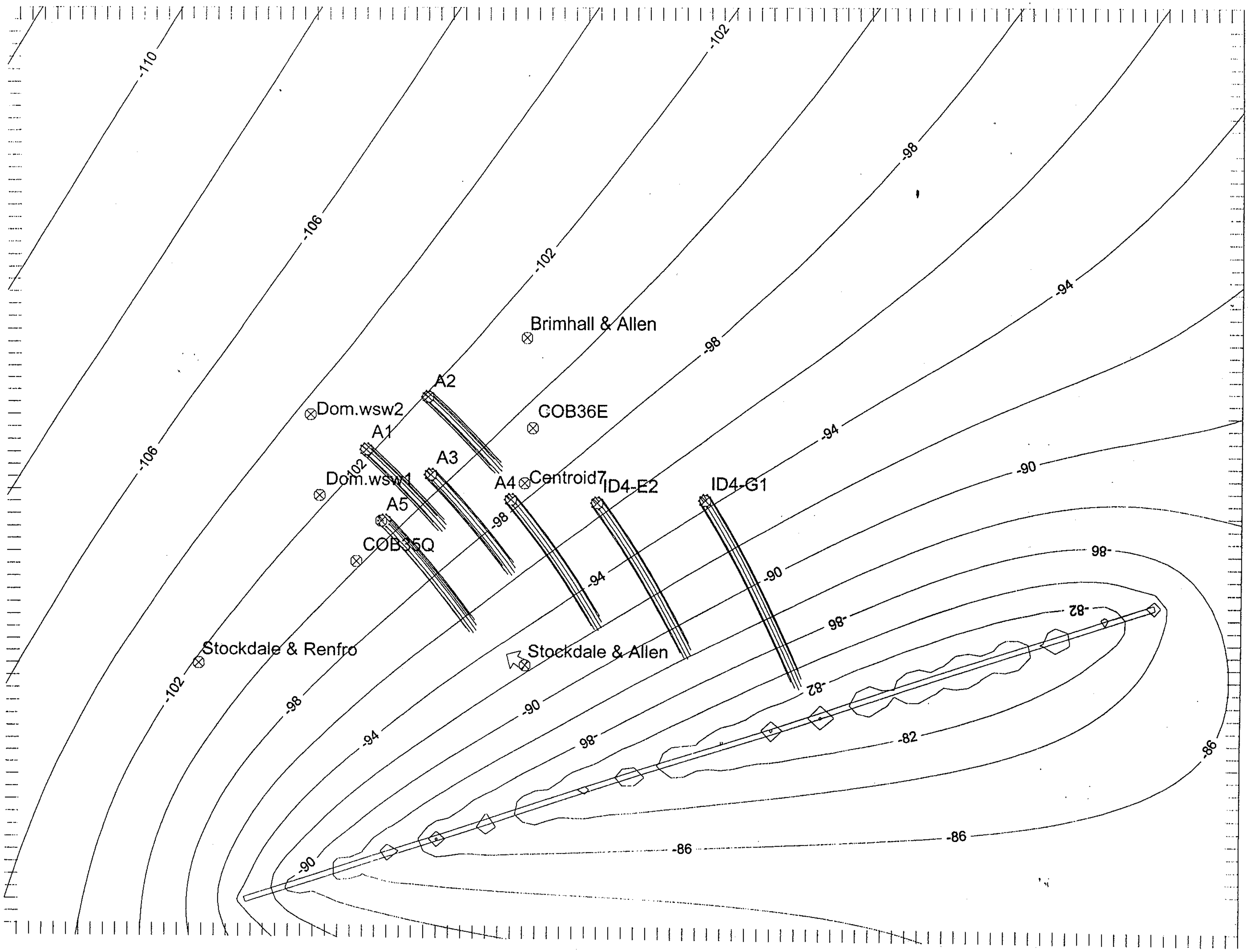


-114
-110
-106
-102
-98
-94
-90
-86
-82
-78
Brimhall & Allen
A2
Dom.wsw2
COB36E
A1
A3
Dom.wsw1
A4
Centroid7
ID4-E2
ID4-G1
A5
COB35Q
Stockdale & Renfro
Stockdale & Allen

223



Brimhall & Allen
COB36E
Centroid7
ID4-E2
ID4-G1
A1
A2
A3
A4
A5
Dom.wsw1
Dom.wsw2
COB35Q
Stockdale & Renfro
Stockdale & Allen
-110
-106
-102
-98
-94
-90
-86
-82


-110
-106
-106
-102
-102
-102
-102
-98
-98
-98
-98
-98
-98
-98
-94
-94
-94
-94
-90
-90
-90
-90
-86
-86
-86
-82
-82
-82
Brimhall & Allen
A2
Dom.wsw2
COB36E
A1
A3
Dom.wsw1
A4
Centroid7
ID4-E2
ID4-G1
A5
COB35Q
Stockdale & Renfro
Stockdale & Allen

Appendix 6.

Discussion

SETS 01 - 06.

Summary of Drawdown Analyses; Sets 1 - 6.

Set 1. Base case and limiting cases.

The aquifer in the area of interest is neither a fully confined aquifer nor a fully unconfined aquifer. The aquifer is a semi-confined aquifer, also referred to as a leaky aquifer, meaning that this aquifer is bounded on the top by a semi-permeable layer (an aquitard) rather than impermeable layer. Furthermore, this semi- confining layer is itself overlain by another water-bearing zone which is a source of recharge water to the aquifer. The aquifer gets recharged from above as well as from the sides, and this is how we have modeled the aquifer in the project area for the calculation of drawdowns.

We can also calculate hypothetical drawdowns as if the aquifer were either fully confined or fully unconfined, all else being equal, as well. It is useful to do so for illustrative purposes because the drawdowns are dramatically different and serves to remind the user of the pitfalls of underestimating the importance of careful model and parameter selection.

Leaky aquifer. In our opinion, this is the computational model that best represents the actual aquifer conditions in the project area. The base case drawdown is for seven wells pumping continuously for 300 days from a leaky aquifer with a parameter set which currently consists of the best available measured values for the required parameters. Under these conditions, the drawdown due to pumping is predicted to decline for only 10 - 20 days before leveling out and remaining steady for an indefinite period, perhaps years, depending on actual recharge.

This steady state condition will persist as long as the rate of aquifer extraction is re-supplied by the rate of downward recharge from the overlying water table, i.e., leakage into the aquifer. This steady state could continue for a long time and represents the limiting condition for a leaky aquifer, as long as recharge keeps up with extraction. It is possible that pumping may never continue long enough to exceed leaky steady state behavior.

If the project area does not receive sufficient recharge, then the shallow zone may be dewatered in which case shallow wells may go dry. If this happens, then leakage will be insufficient to re-supply the rate of extraction, at which time the pumping will continue to remove water from aquifer storage and the head will continue to decline. If the confined aquifer were laterally infinite, then the head would continue to decline forever. But the project area is relatively close to an upgradient recharge boundary so that even drawdown in the confined aquifer would ultimately stabilize at some lower head.

Confined aquifer. This computational model has been used by other workers in previous investigations but, in our opinion, it does not represent the observed historical - and expected future- behavior of the aquifer in the project area. This hypothetical case is for seven wells pumping continuously for 300 days from a fully confined aquifer with the same parameter set except that there is no leakage and all water comes from aquifer storage. Under these conditions, the drawdown due to pumping is predicted to decline forever in an infinite aquifer. The drawdown after a pumping duration of 300 days is about 90% of the total drawdown that would be observed after a pumping period of 3 years, so for any of the proposed operating scenarios this essentially represents the confined aquifer limiting case.

Unconfined aquifer. This computational model is not representative of the observed historical- or expected short- term- behavior of the aquifer in the project area under the project assumption of ongoing recharge but, in our opinion, may represent the long- term behavior of the aquifer if long- term total recharge does not keep up with total recovery in the greater project area. This hypothetical case is for seven wells pumping continuously for 300 days from a fully unconfined aquifer with the same parameter set except that there is no aquitard and all water comes from dewatering the aquifer. Under these conditions, the drawdown due to pumping is also predicted to decline forever in an infinite aquifer. Like the previous case, the drawdown after a pumping duration of 300 days is about 90% of the total drawdown that would be observed after a pumping period of 3 years, so for any of the proposed operating scenarios this essentially represents the unconfined aquifer limiting case.

The aquifer in the project area is not currently an unconfined aquifer but it could become one if the shallow aquifer is dewatered and/or insufficiently recharged. If the actual aquifer conditions start out in the leaky aquifer condition and then change to an unconfined condition, then the observed drawdowns will no longer remain at the leaky steady state static levels but will continue to decline in the project area until the predicted unconfined drawdowns have been achieved.

Set 2. Variation of base case drawdown with leakage rate (B) for t = 300d.

The rate of leakage through the aquitard determines the rate of aquifer recharge and this leakage rate is incorporated mathematically into the model through the Hantush leakage factor (B). Higher leakage rates (lower B values) cause the drawdown to stabilize at the steady state value more quickly and at a smaller final drawdown. Conversely, lower leakage rates (higher B values) cause the aquifer to behave more like a confined aquifer in which drawdown lasts longer and is larger when it finally stabilizes.

We have calculated the drawdowns for the other base case parameters and for the set of leakage values of B = 1800, 2200, 2600, 3200, 5000, 10000. For example, a leakage factor of B = 2,000 might represent a thin, more transmissive aquitard perhaps 20 ft thick aquitard with a Kv = 0.1 ft/d, while a leakage factor of B = 10,000 might represent a thicker, less transmissive aquitard perhaps 50 ft thick aquitard with a Kv = 0.05 ft/d.

As previously discussed, the overall aquitard leakiness of this particular project area is neither that of the silts alone nor of the interbedded sands alone, but a composite intermediate value of both which cannot be determined from the limited E-log data in the project area. The leakance of the aquitard is complicated by the unretarded vertical flow at the edges of a localized silt/clay layer. The bypass flow makes an aquitard look leakier than the measured parameters indicate, but this is very difficult to evaluate. The discontinuous nature of the aquitard contributes an unpredictable component of non-ideal behavior to the results which we cannot model, but some of the observed differences between predicted and actual drawdown behavior which we anticipate will be observed once the project has begun will likely be due to different rates of leakage recharge at different pumping wells based on the non-ideal particular properties of the overlying aquitard at those locations.

Set 3. Variation of base case drawdown with time (t).

Based on the base case parameter values, we calculate that drawdown will stabilize at the steady state maximum drawdown after 10 - 20 days. This means that as long as the shallow water table aquifer continues to supply the underlying semi-confined aquifer with recharge water, the aquifer drawdown *will remain constant*, even for three consecutive years of pumping. The accuracy of this hypothetical forecast depends mostly on whether or not sufficient water is recharged into the project area to sustain the recovery volumes over time. We have calculated the drawdowns for the base case parameters and for the set of pumping times of t = 10, 30, 100, 300, 720 days. The calculation for t = 720 d represents the 300 recovery days per year for 3- consecutive year operating scenario after being corrected for the effect of two months recovery time in between pumping periods.

Set 4. Variation of base case by varying pump duration (t) at $Q = Q_{full}$.

In any year for which the amount of groundwater to be recovered is less than the project design capacity of 27,000 af over a 300 day period, the operator has several options. The operator might pump all wells at the design pump rate but for a shorter time period. The operator might pump all wells for the full period but at a lower pump rate. The operator might also pump fewer wells at any combination of rates and durations which will supply the required recovery volume. The evaluation of the secondary factors which might govern such a decision are outside the scope of this analysis but they might include scheduling priorities or capacity availability in conveyance systems, differing well efficiencies and lifting costs, water quality issues, etc. For our scope of work, we have evaluated the drawdown scenarios for 50% and 25% of the 100% base case recovery volume of 27,000 af.

Before we present the results, we can state certain qualitative differences in the amount of drawdown associated with each of these choices based on applicable flow theory. For a given recovery volume where the choice is to either cut the pumping time in half or cut the flow rate in half, the drawdown is always significantly smaller by cutting the flow rate and increasing the time correspondingly. For a given recovery volume where the choice is to either use all wells or fewer wells, the drawdown is always somewhat smaller by using all wells because the extraction is distributed over a somewhat larger pumping area.

The first set of illustrative variations is to operate all wells at full design pump rates for pumping durations of t = 300, 150, & 75 days to recover 100%, 50%, and 25% respectively of the base case recovery volume of 27,000 af. As we have previously presented, the drawdowns for all three cases are exactly the same because all three of these pumping durations exceed the time required to achieve leaky steady state. Therefore, shortening the pumping time does not lessen the amount of drawdown. What is not illustrated in these scenarios is that since pumping lasts for a shorter time period, the drawdown also lasts for a shorter time period before the aquifer begins to recover. For anyone who might operate a domestic water supply well in the zone of impact, two months of extra lift is easier to bear than 5 or 10 months.

Set 5. Variation of base case by varying flow rate (Q) for t = 300d.

The second set of illustrative variations is to operate all wells for the full 300 day recovery period but at lower pumping rates of Q = 50% or 25% of Q_{full} to recover 50% or 25% respectively of the base case recovery volume. Although the pumps will run at lower efficiency, the payoff is big in terms of reduced drawdown impacts. If the wells are operated at x% of base case, then the drawdowns will be x% of the base case drawdowns! So, by operating at 50% of full Q, the drawdowns will be 50% of the base case drawdowns, although they will last for the same length of time. Therefore, reducing the pumping rate causes significant reductions in the amount of drawdown at all locations and for any pumping period.

Set 6. Variation of base case by pumping only selected wells (t = 300d).

The third set of illustrative variations is to operate some but not all wells for the full 300 day recovery period. We have selected just three of many possible cases for illustration. One case is to pump just the five RRB wells at full pump rate of 5 cfs for a 300 day recovery period which would supply about 56% of the full recovery capacity. A second case is to pump just the two ID4 wells at full pump rate of 10 cfs for a 300 day recovery period which would supply about 44% of the full recovery capacity. The third case is to pump one RRB well and one ID4 well at their respective design rates for a 300 day recovery period which would supply about 33% of the full recovery capacity. For this third case, we selected the two wells which are the farthest apart, i.e., at opposite ends of the east - west trending well field.

Because the first two cases add up to 100% of the full base case, these two cases represent the respective components of the total drawdown for the wells west of Allen Rd (case 1, RRB wells) and east of Allen Rd (case 2, ID4 wells). Divided in this way, either grouping can supply approximately 50% of the base case recovery volume and the drawdowns from each are centered on significantly different parts of the project area.

The third case illustrates the overlapping impacts of two pumping wells which are about 5,500 ft apart after 300 days of pumping. The westerly well pumps at 5 cfs while the easterly well pumps at 10 cfs. The differences in drawdowns are most pronounced in the vicinity of each well, where the drawdowns of the 10 cfs well are twice that of the 5 cfs well. The drawdowns become more nearly alike at the farther radial distances from each well as the drawdowns decrease logarithmically with distance out to the edge of the zone of influence. These two wells can supply about 33% of the base case recovery volume.

There are numerous other possible operating scenarios to recover any amount of water within the project design capacity, all of which involve a choice of wells, pumping rates, and pumping durations. An evaluation of other such scenarios is outside the scope of work, but can be completed, if desired, as an extension of this work program.

Appendix 6.

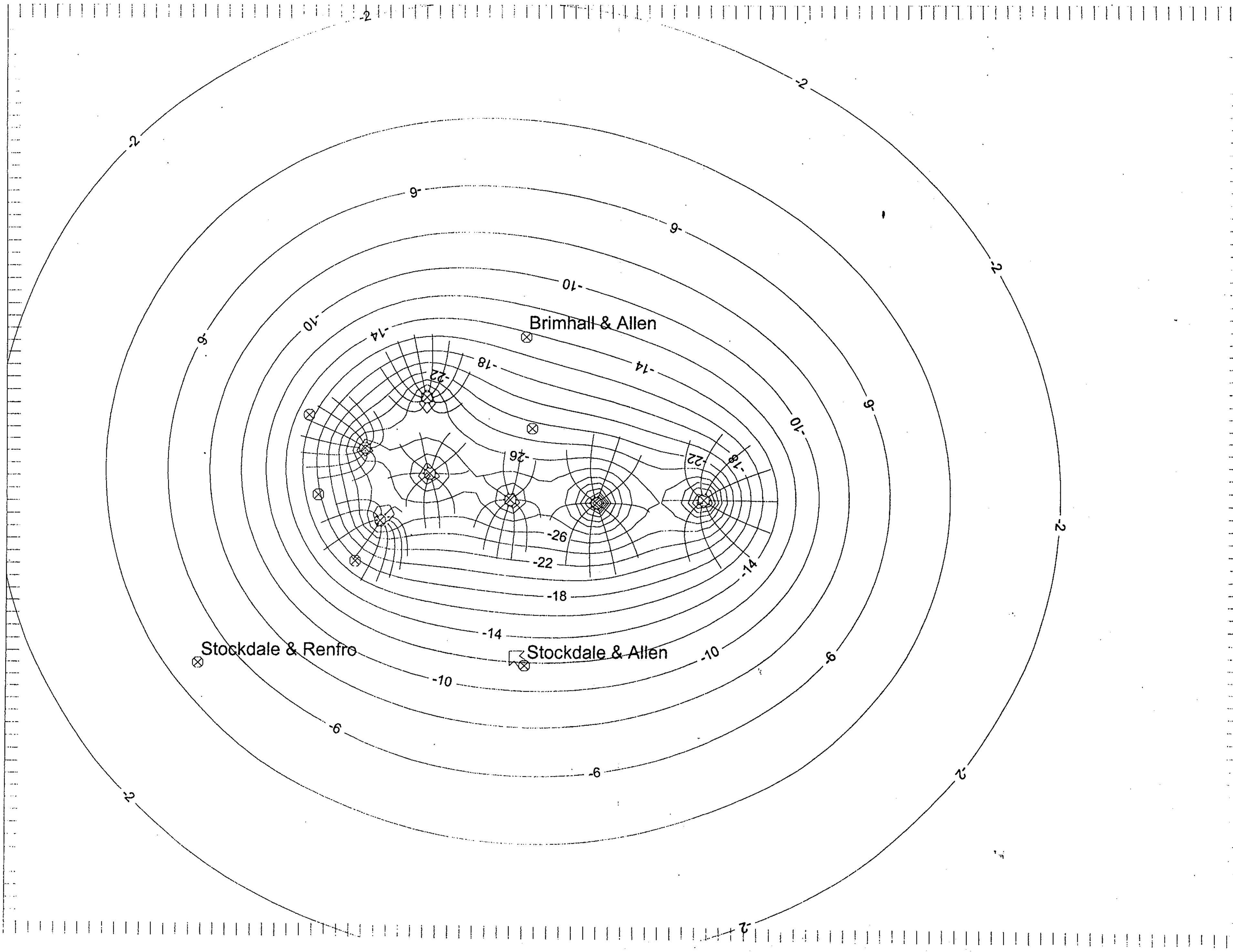
SET 01.

ID4 / KT / RRB Well Field Model Parameters.

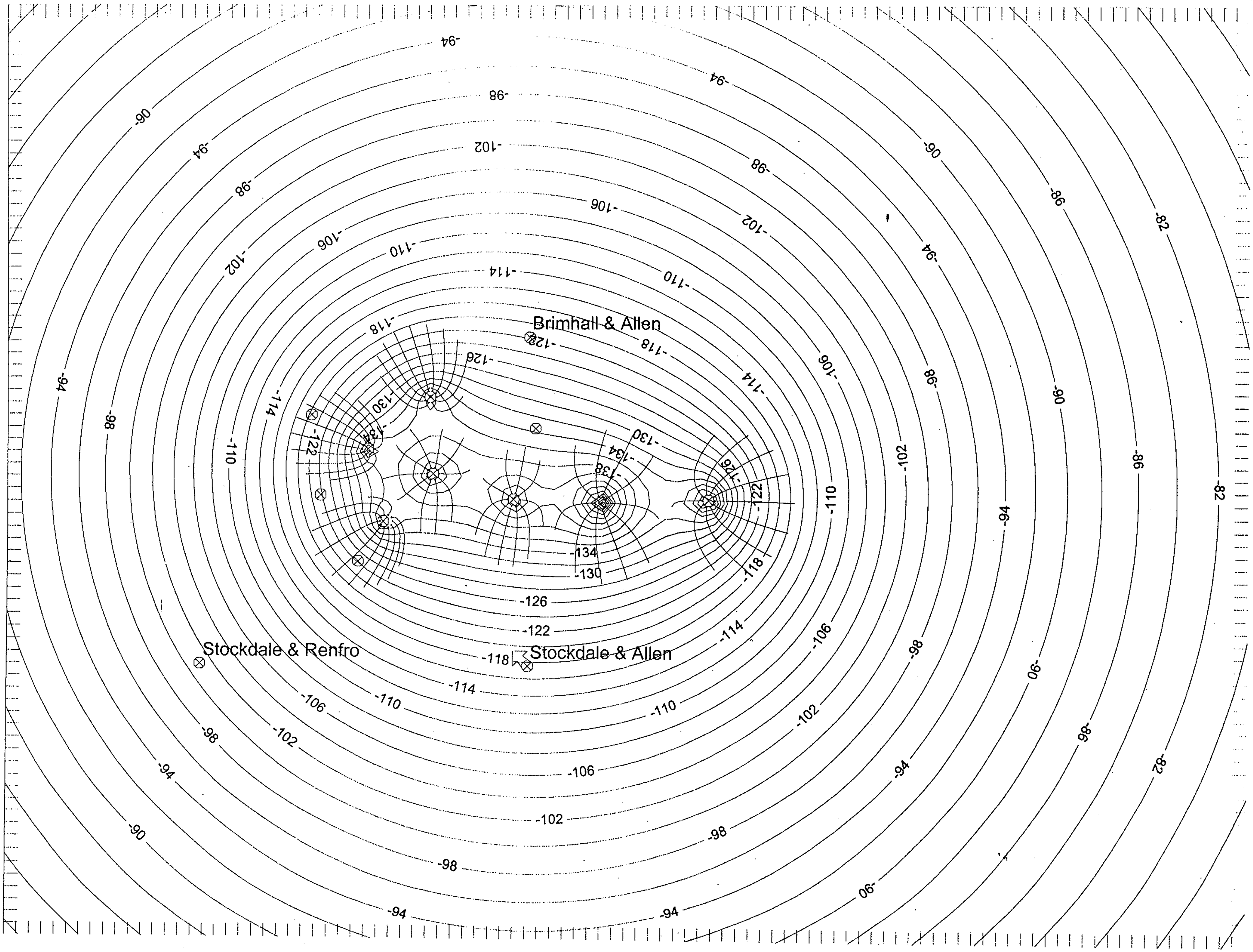
Set No. 1.

Note: only CHANGES are tabulated between Runs, from left to right.
Note: once made, CHANGES persist until modifed again.

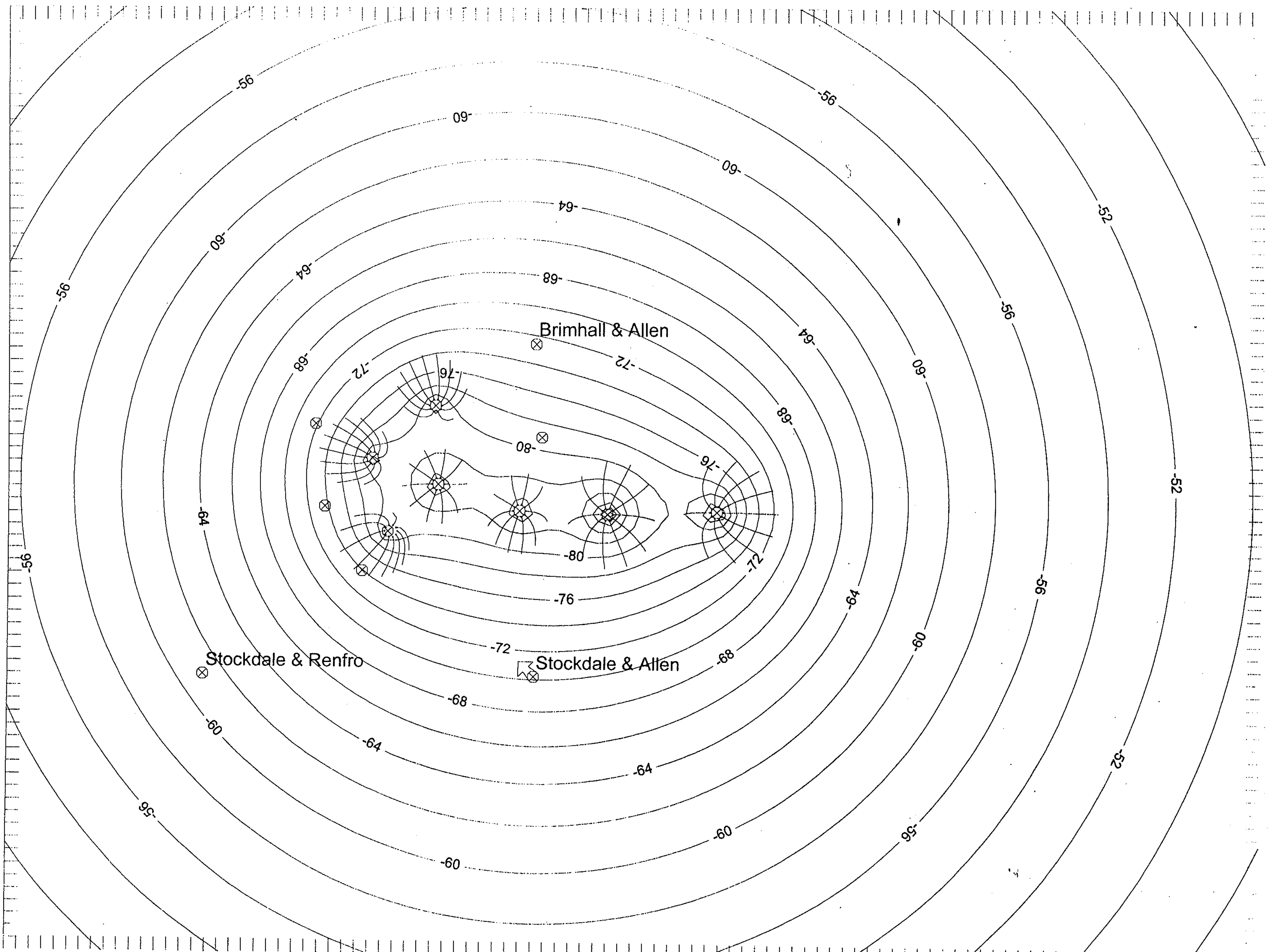
		base case												
model		leaky	confined	uncon	uncon	uncon	uncon	confined	leaky					
calc		transient	transient	transient	transient	transient	transient	transient	transient					
Run No.		101	102	103	104	105	106	107	108					
H.Cond.	K	80												
Btm	B	-450												
Top	T	-200		1				-200						
Ref Hd	R	0												
Grad	G	0												
Rchg	Rch	0												
Por	P	0.3												
Stor	S	5.60E-04			0.21		5.60E-04							
Lkg	L	3200	0						3200					
pump-t	t	300				770								
Ref Hd	Ref	0												
Grad	G	0												
Azim	A	135												
Ref-X	X	0												
Ref-Y	Y	0												
Const Hd	CH	no												
trace-t	tt	300												
Flow Rate	Q	full												
Wells	W	all				770								



Brimhall & Allen
Stockdale & Allen
Stockdale & Renfro
-2
-6
-10
-14
-18
-22
-26

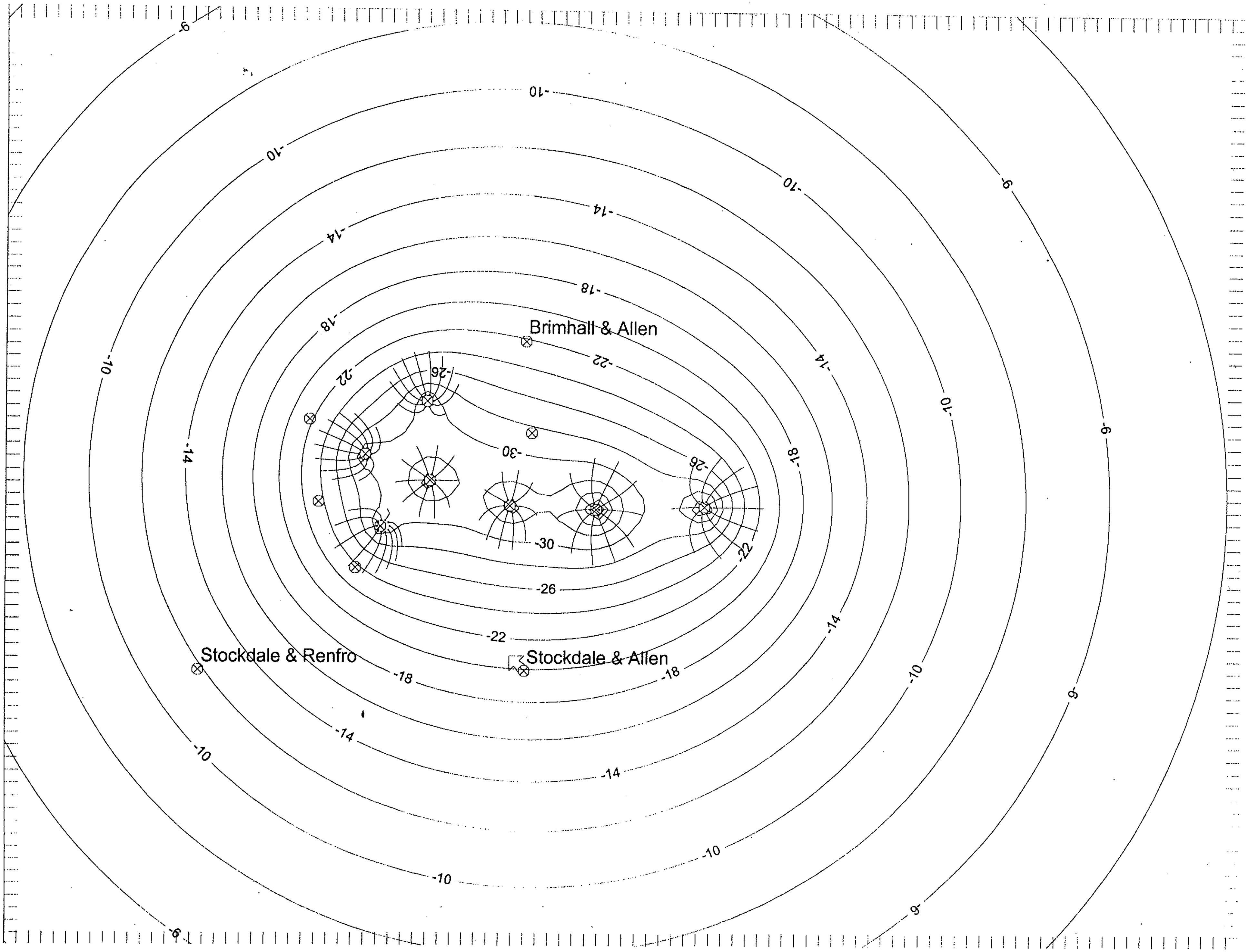


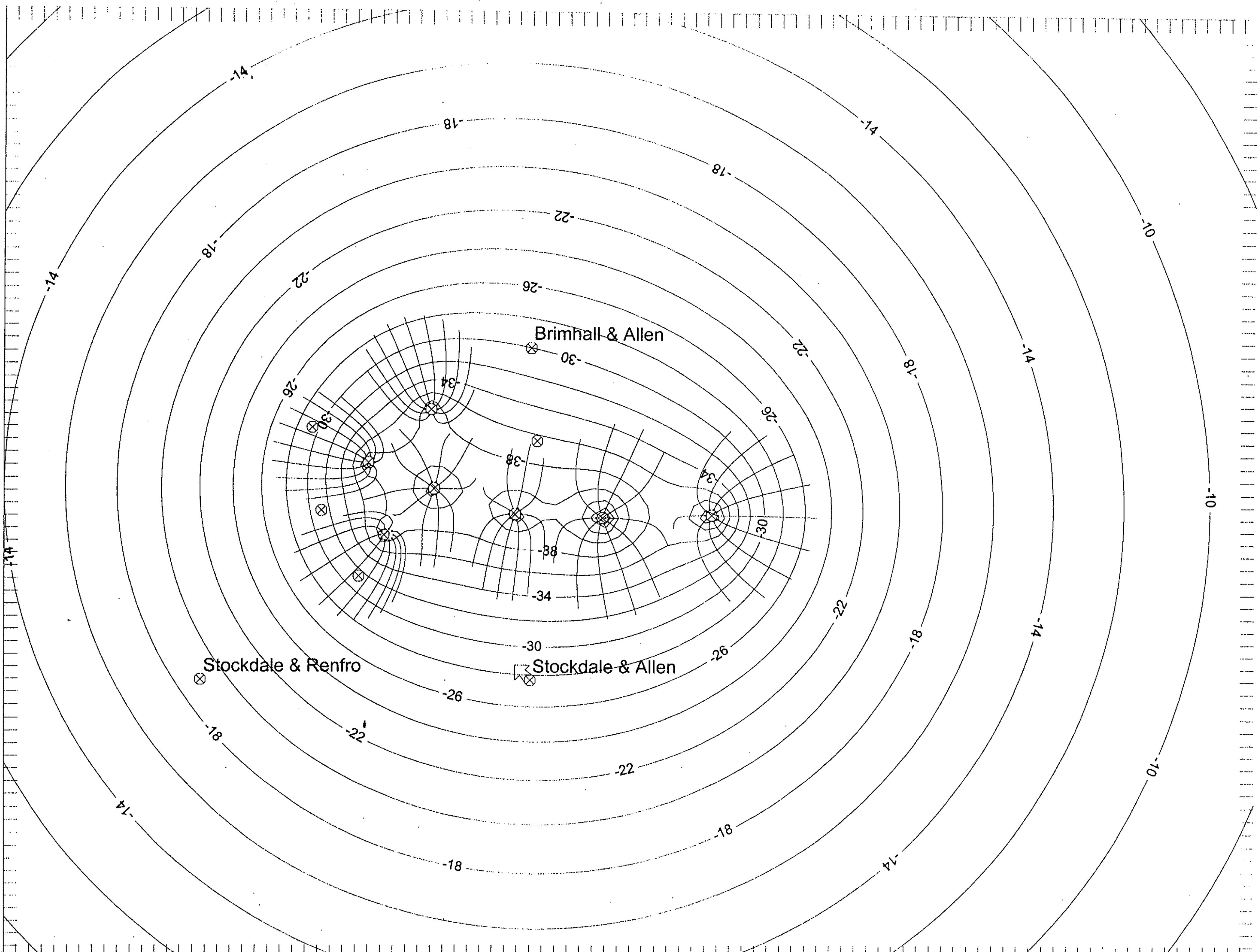
Brimhall & Allen
Stockdale & Allen
Stockdale & Renfro



Brimhall & Allen
Stockdale & Allen
Stockdale & Renfro
-52
-56
-60
-64
-68
-72
-76
-80

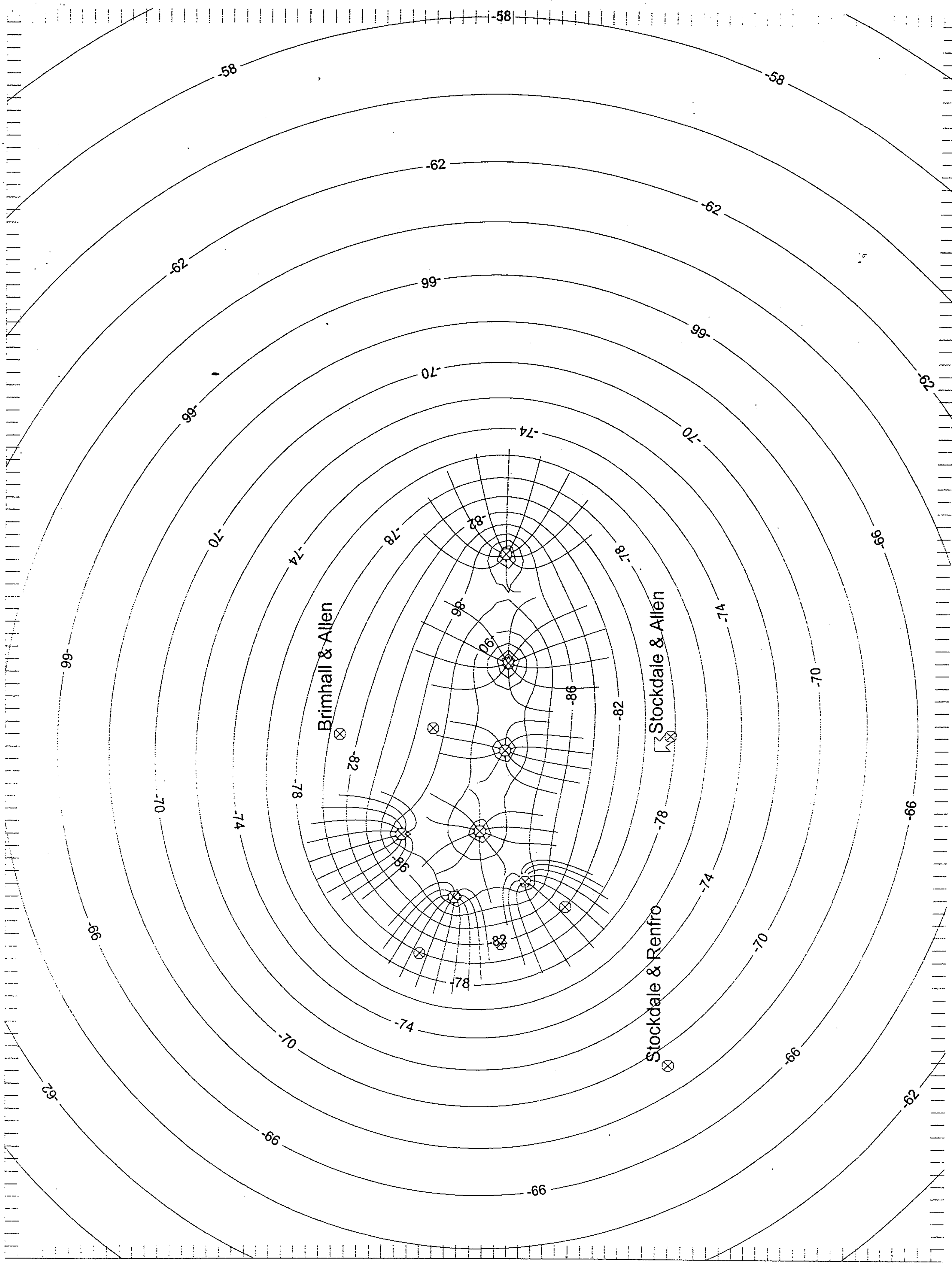
107

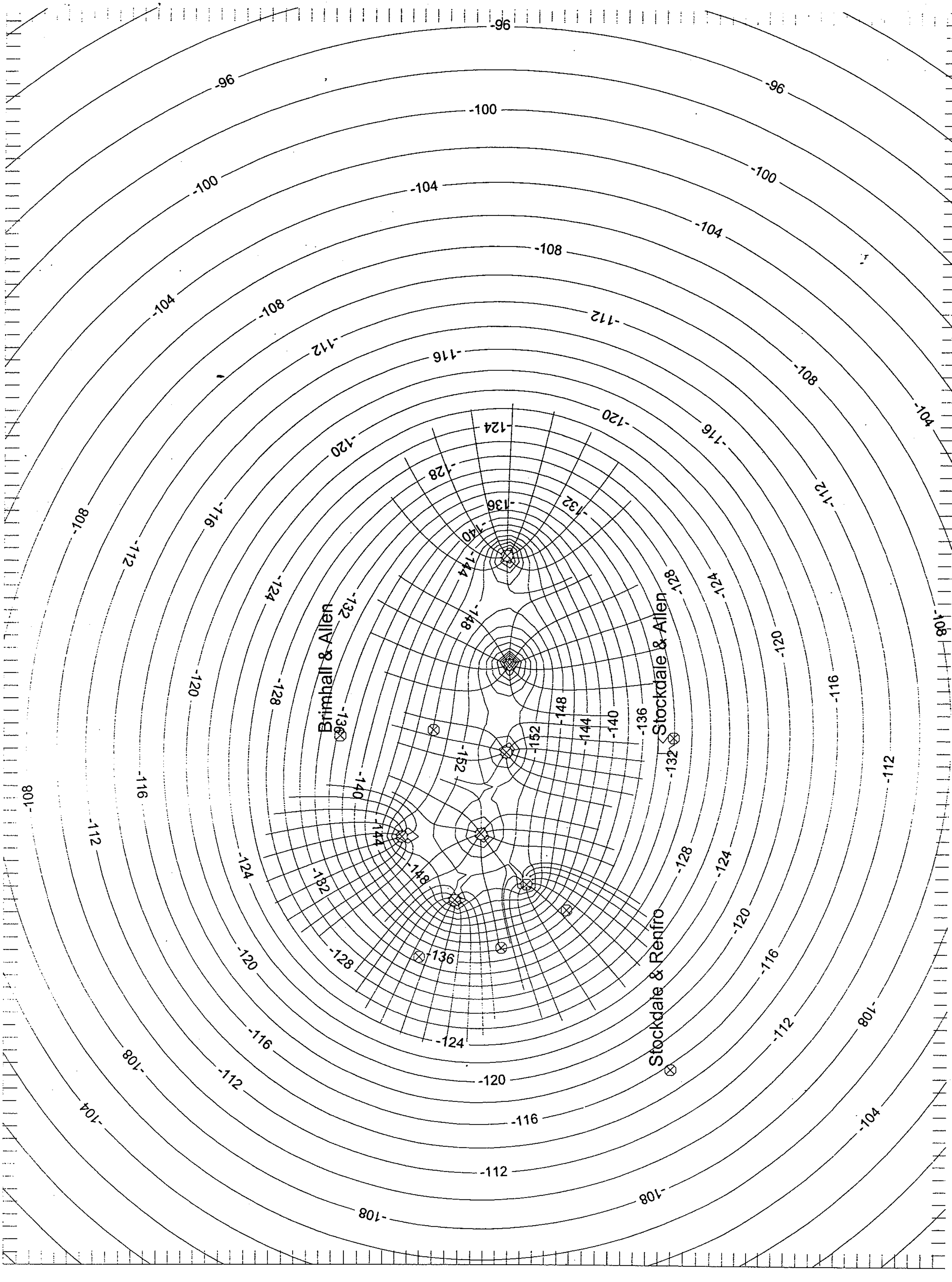

Brimhall & Allen
Stockdale & Renfro
Stockdale & Allen
-6
-10
-14
-18
-22
-26
-30

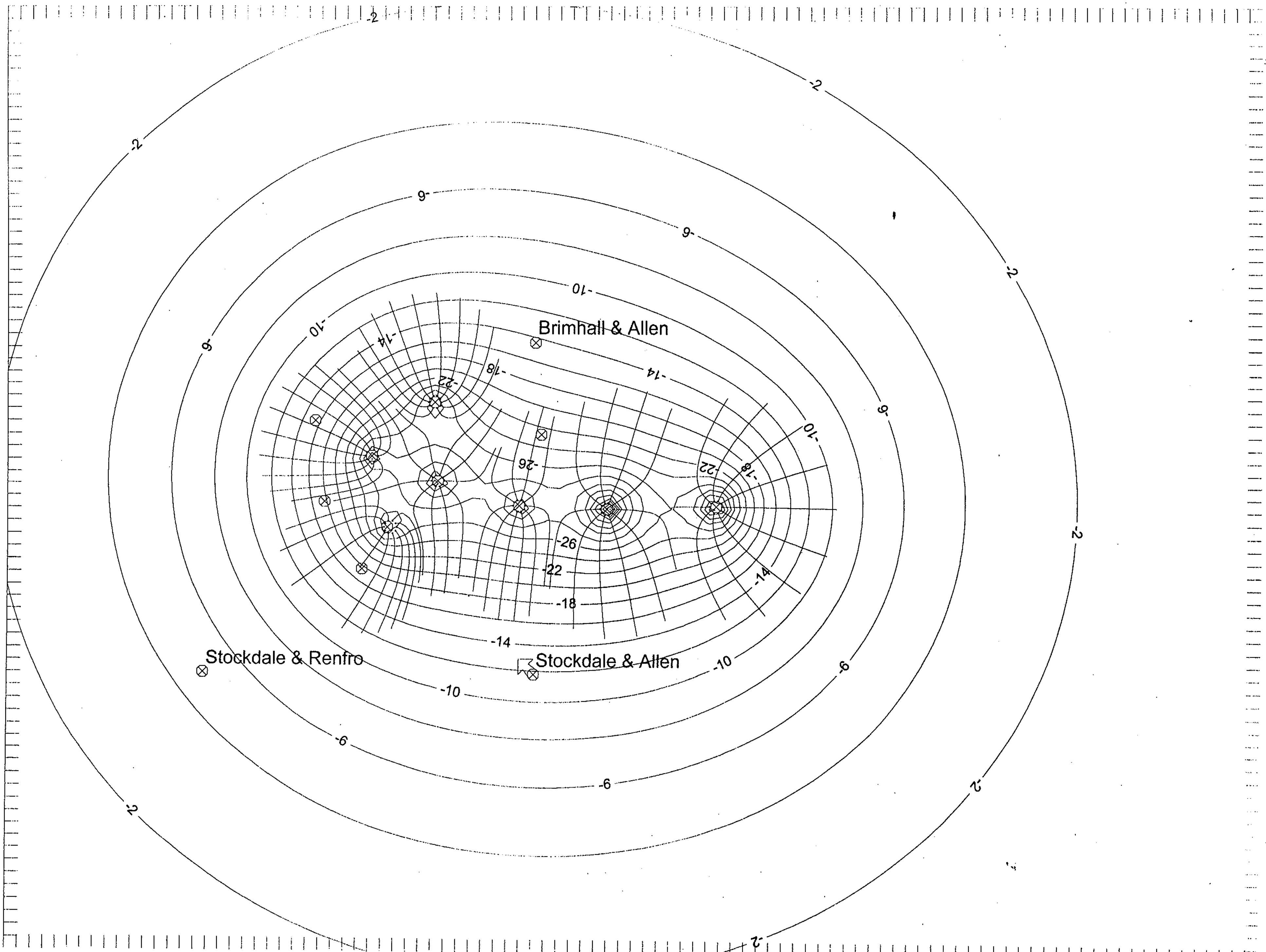


Brimhall & Allen
Stockdale & Renfro
Stockdale & Allen
-10
-14
-18
-22
-26
-30
-34
-38

105


Brimhall & Allen
Stockdale & Allen
Stockdale & Renfro
-58
-62
-66
-70
-74
-78
-82
-86
-90


Brimhall & Allen
Stockdale & Allen
Stockdale & Renfro



Brimhall & Allen
Stockdale & Allen
Stockdale & Renfro

Appendix 6.

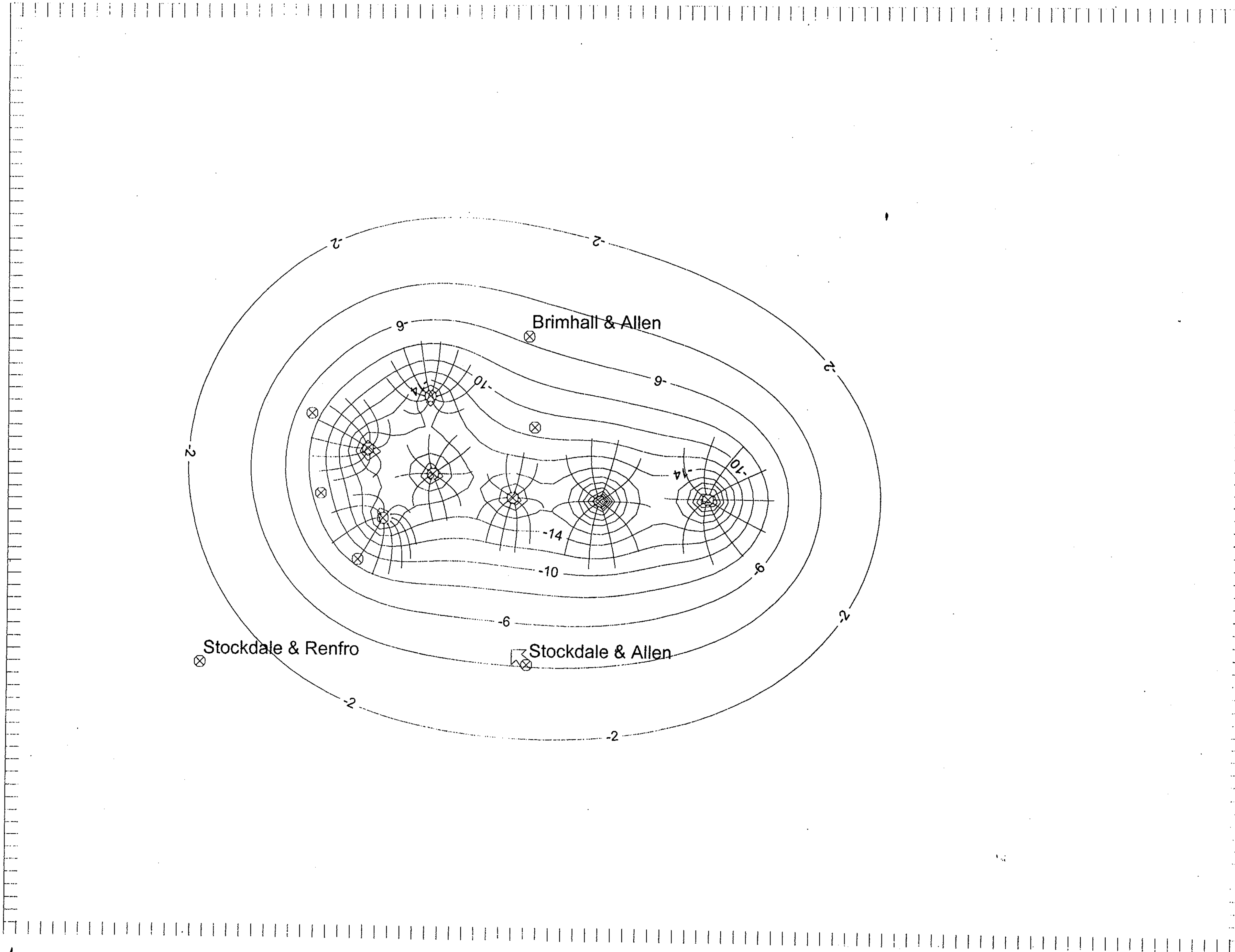
SET 02.

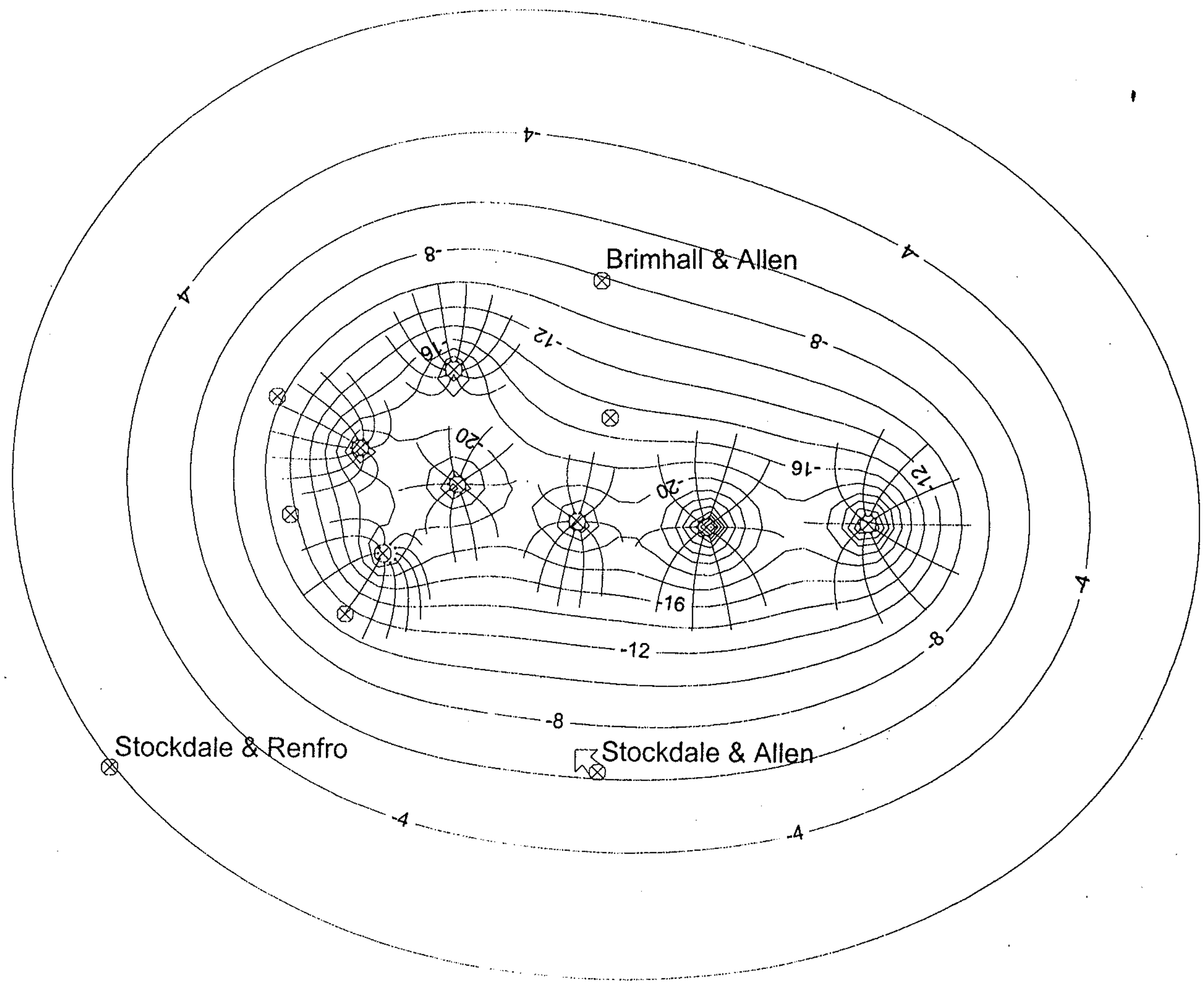
ID4 / KT / RRB Well Field Model Parameters.

Set No. 2.

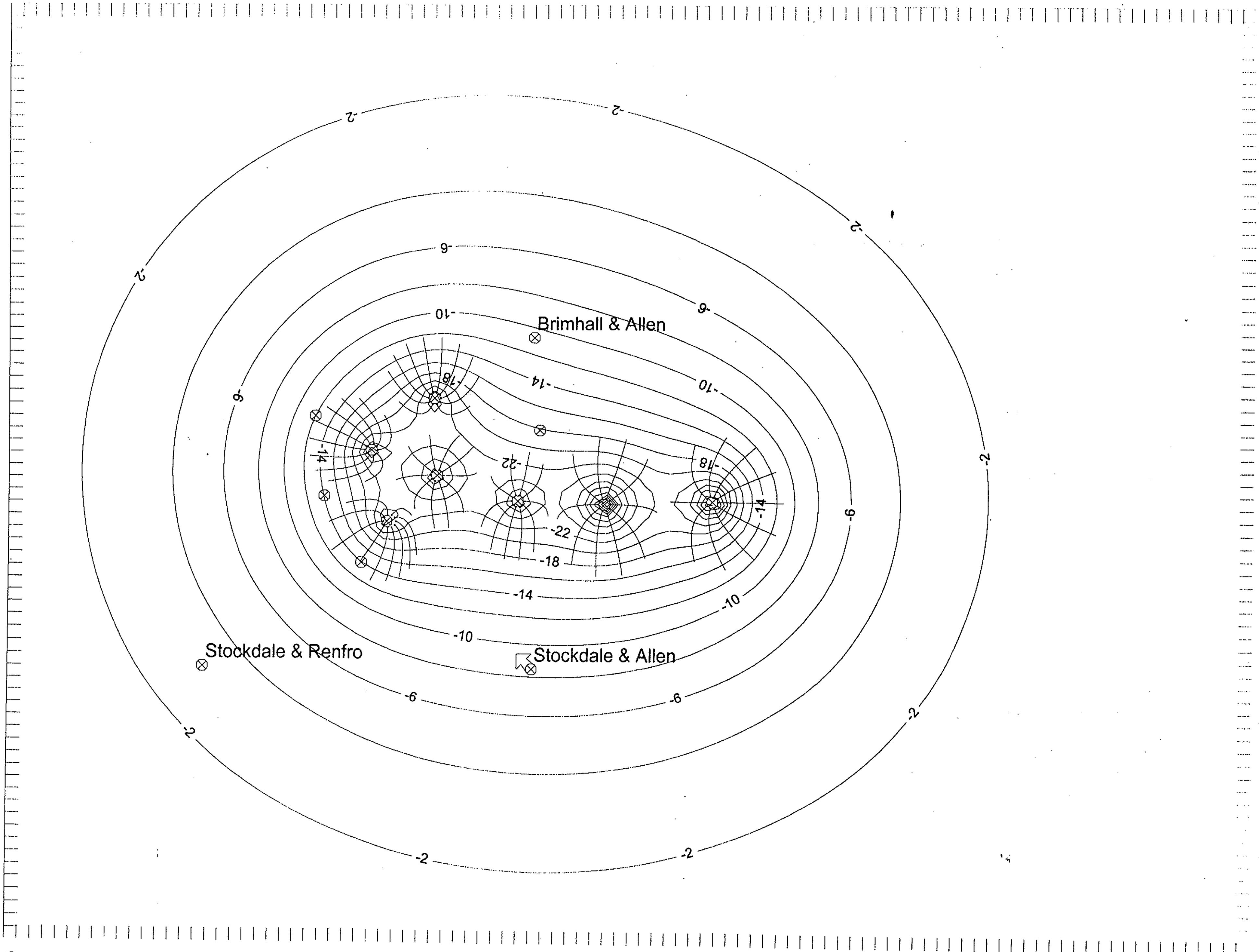
Note: only CHANGES are tabulated between Runs, from left to right.
Note: once made, CHANGES persist until modifed again.

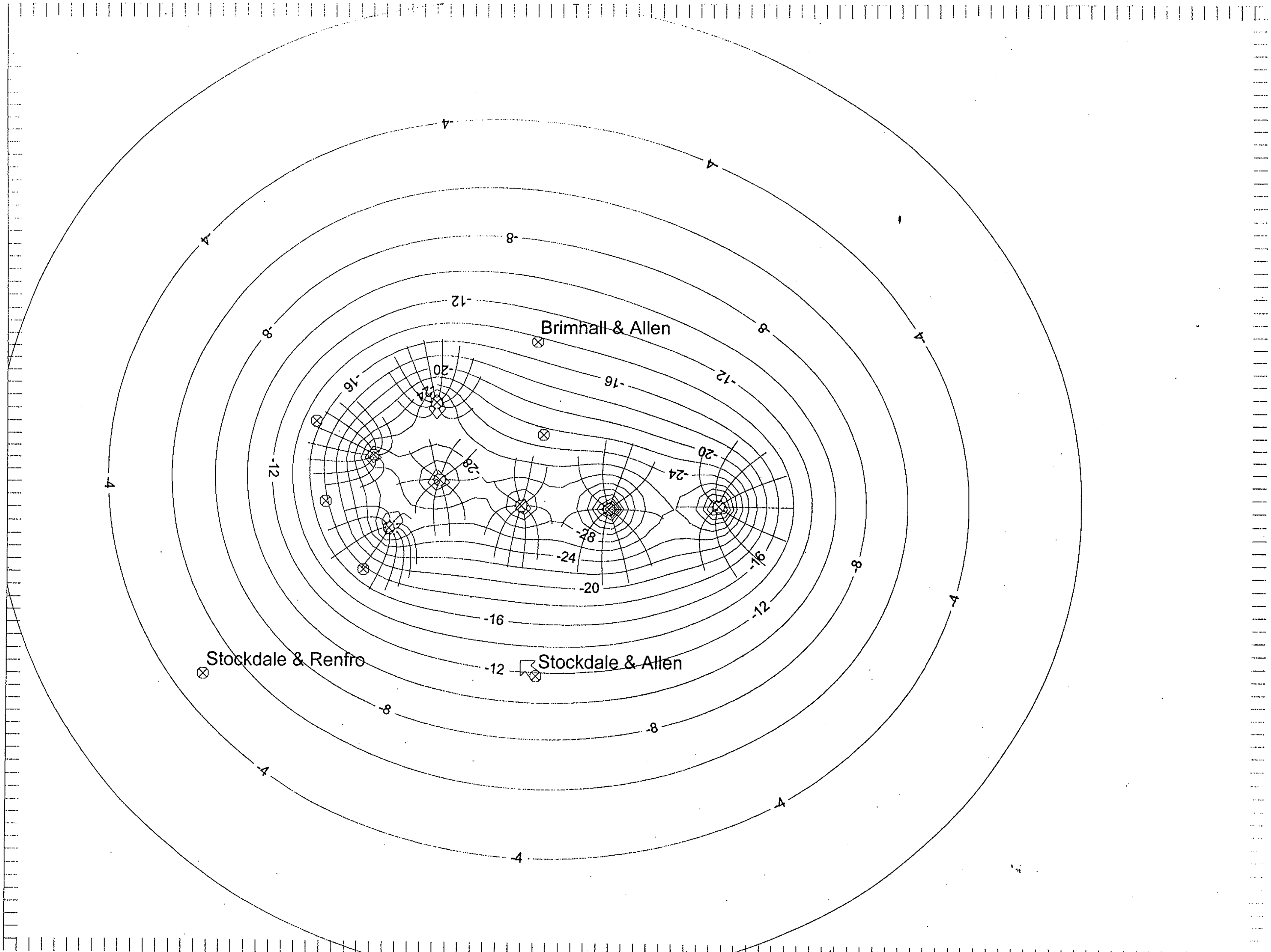
		base case												
model		leaky	leaky	leaky	leaky	leaky	leaky	leaky	leaky					
calc		transient	transient	transient	transient	transient	transient	transient	transient					
Run No.		ref.only	201	202	203	204	205	206	207					
H.Cond.	K	80												
Btm	B	-450												
Top	T	-200												
Ref Hd	R	0												
Grad	G	0												
Rchg	Rch	0												
Por	P	0.3												
Stor	S	5.60E-04												
Lkg	L	3200	1800	2200	2600	3200	4600	6000	10000					
pump-t	t	300												
Ref Hd	Ref	0												
Grad	G	0												
Azim	A	135												
Ref-X	X	0												
Ref-Y	Y	0												
Const Hd	CH	no												
trace-t	tt	300												
Flow Rate	Q	full												
Wells	W	all												

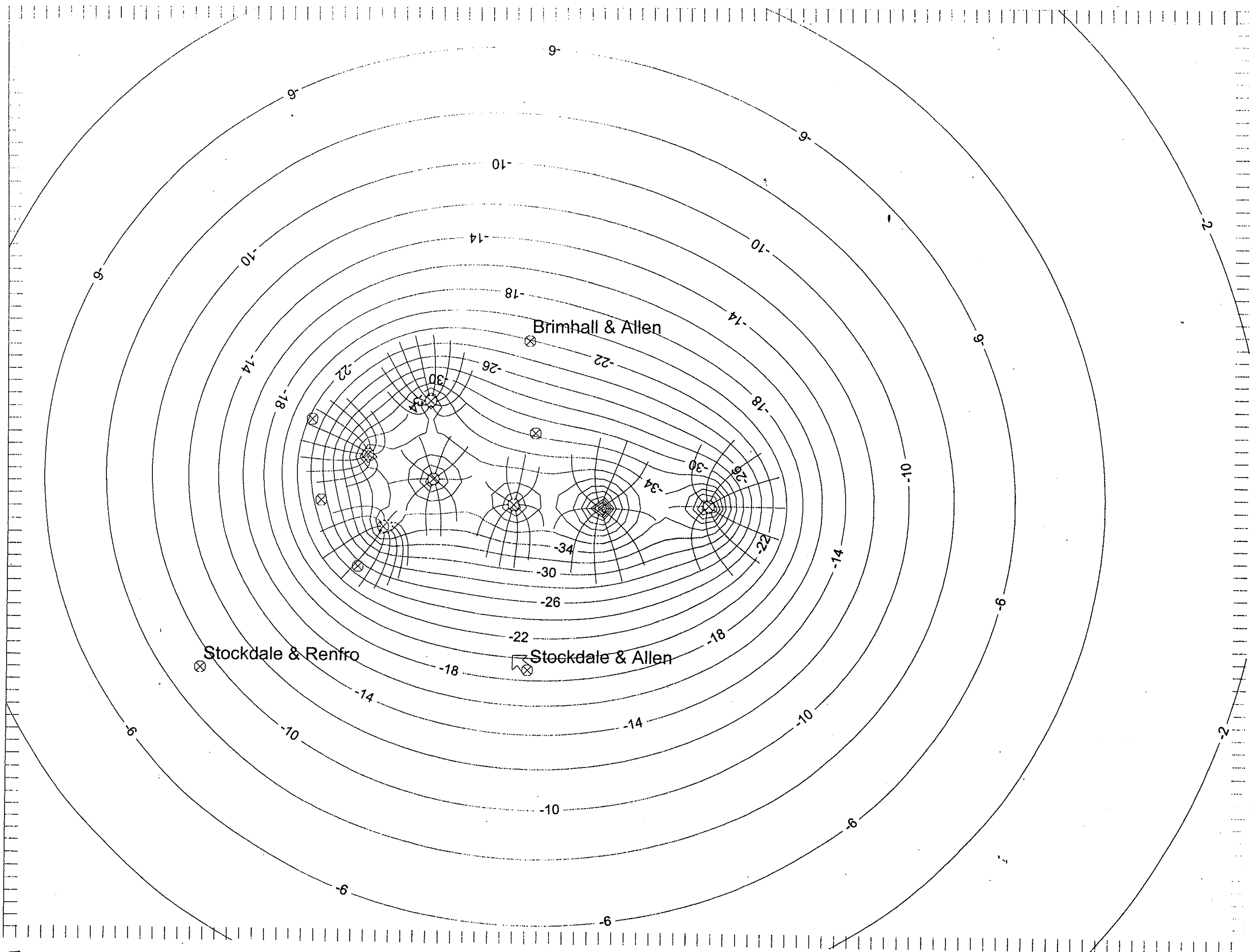

-2
-6
-10
-14
Brimhall & Allen
Stockdale & Allen
Stockdale & Renfro

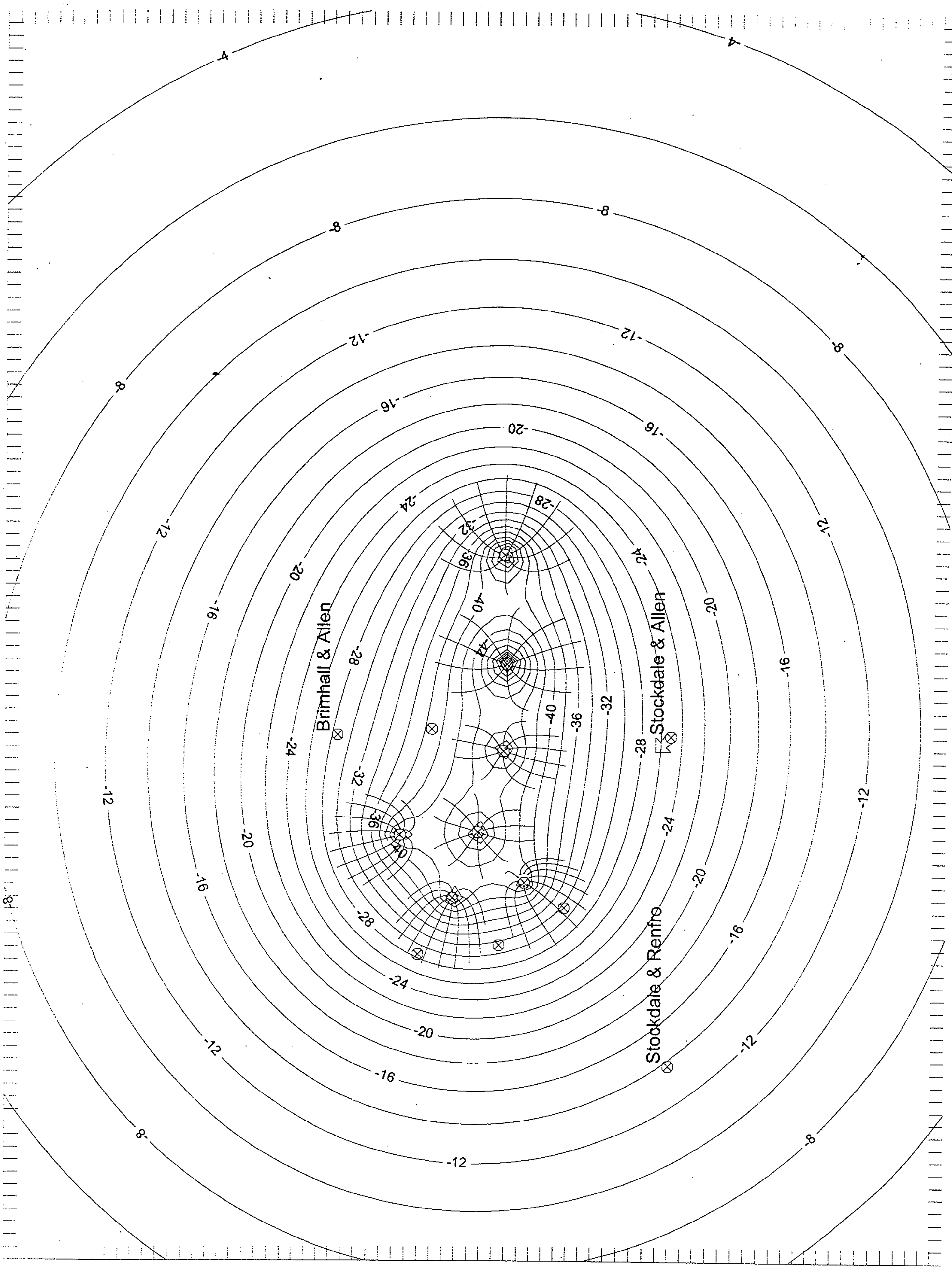


Brimhall & Allen
Stockdale & Renfro
Stockdale & Allen

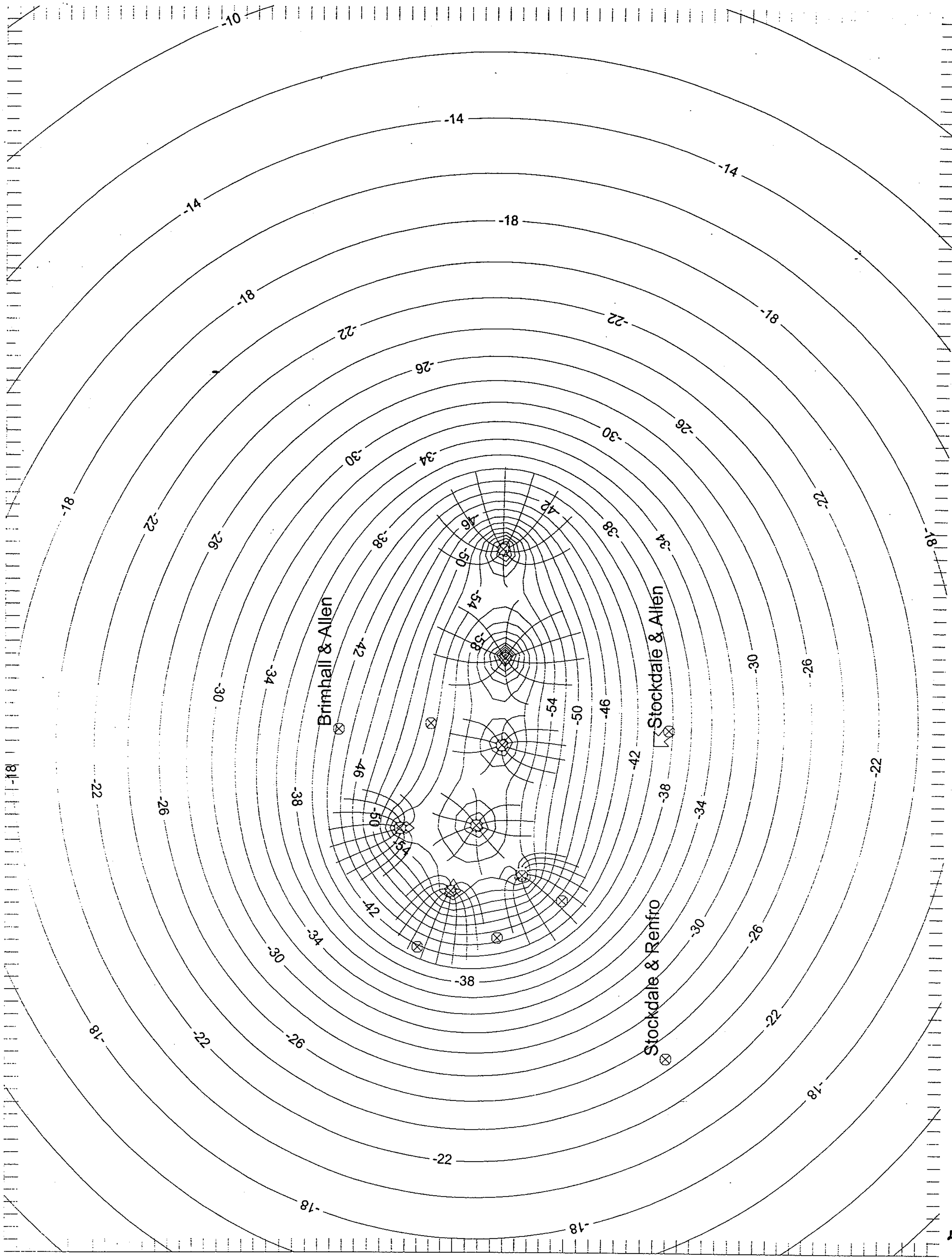

Brimhall & Allen
Stockdale & Renfro
Stockdale & Allen
-2
-6
-10
-14
-18
-22


Brimhall & Allen
Stockdale & Renfro
Stockdale & Allen
-4
-8
-12
-16
-20
-24
-28


Brimhall & Allen
Stockdale & Renfro
Stockdale & Allen
-2
-6
-10
-14
-18
-22
-26
-30
-34



Brimhall & Allen
Stockdale & Allen
Stockdale & Renfro



Brimhall & Allen
Stockdale & Allen
Stockdale & Renfro
-10
-14
-18
-22
-26
-30
-34
-38
-42
-46
-50
-54
-58

Appendix 6.

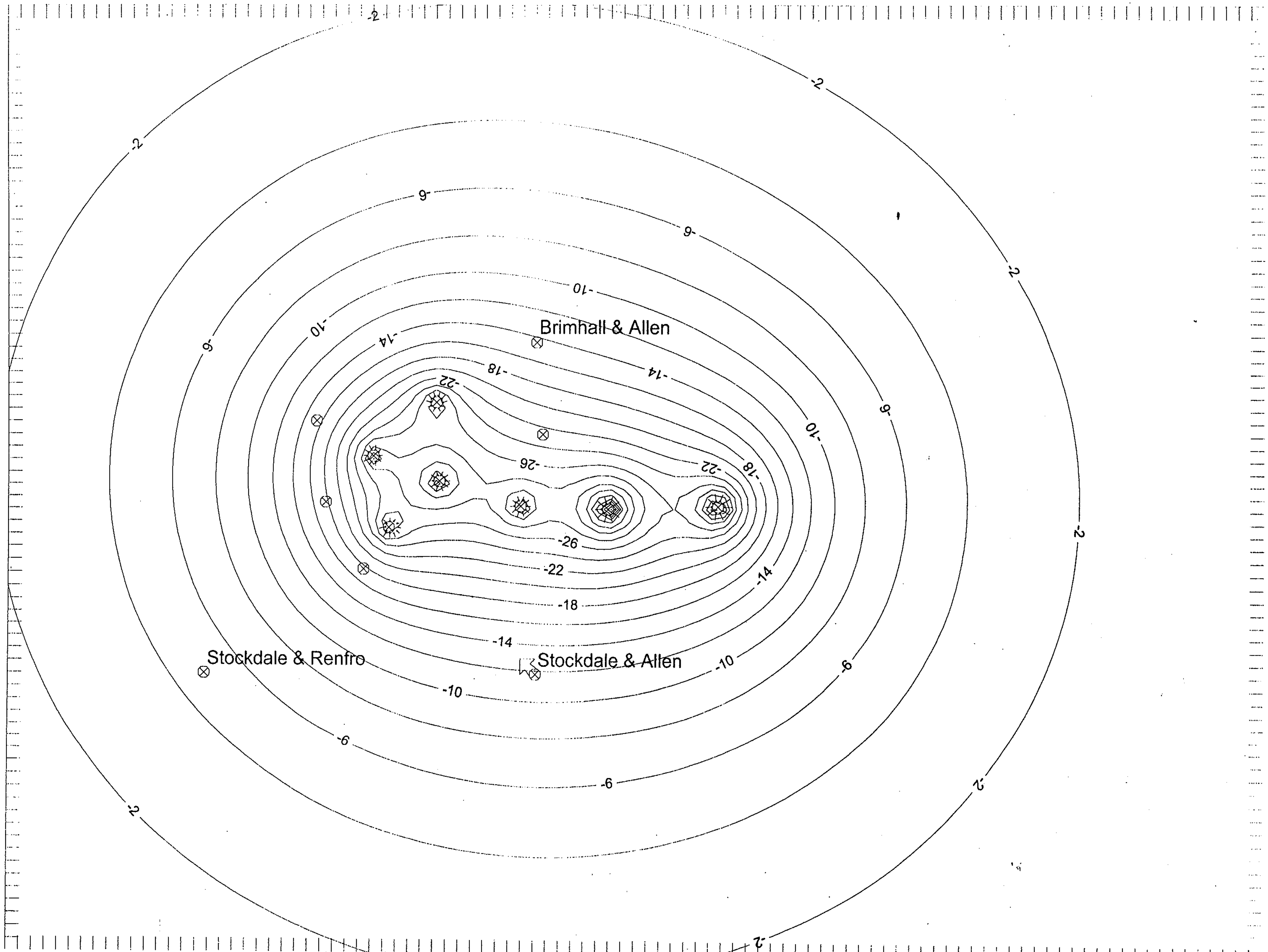
SET 03.

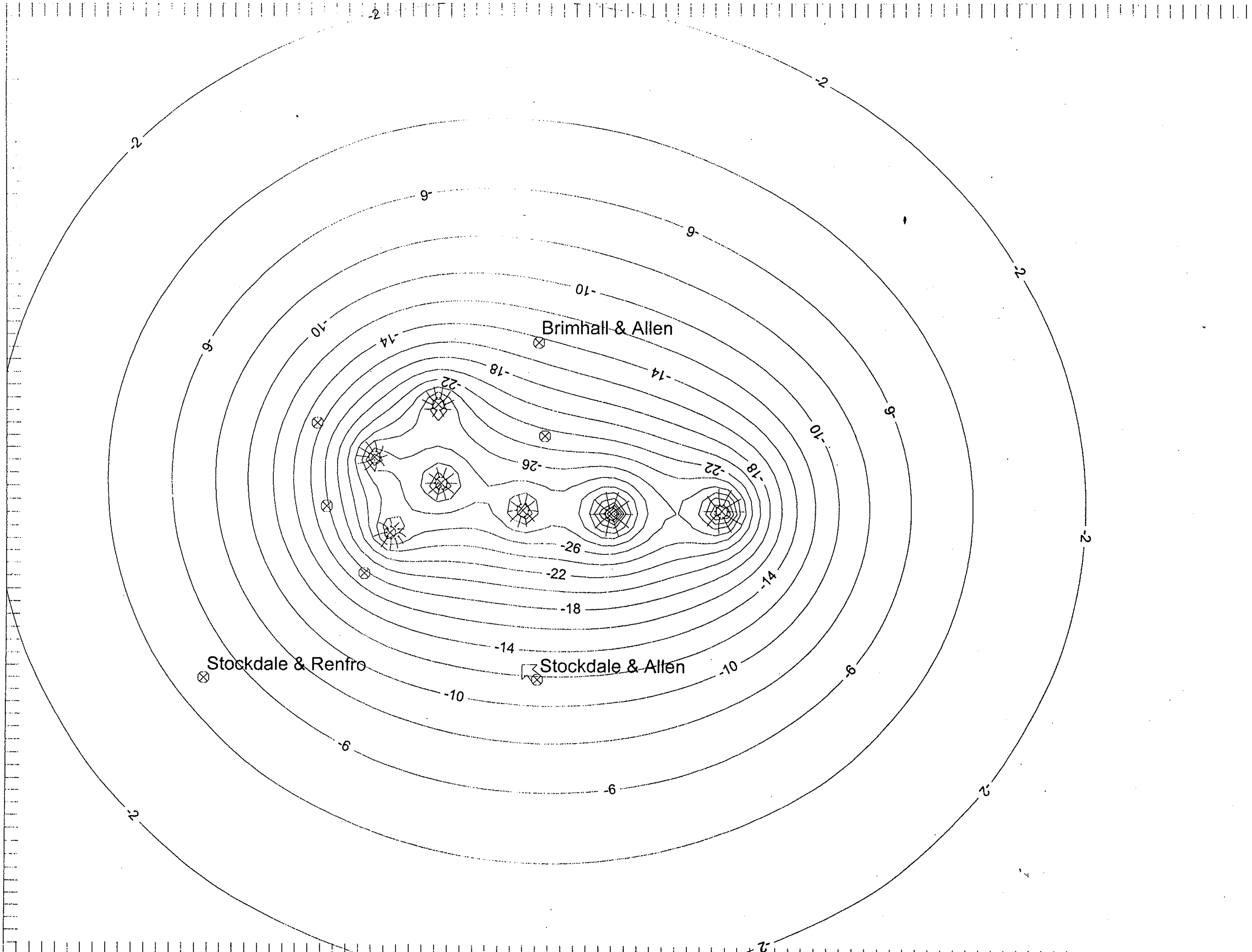
ID4 / KT / RRB Well Field Model Parameters.

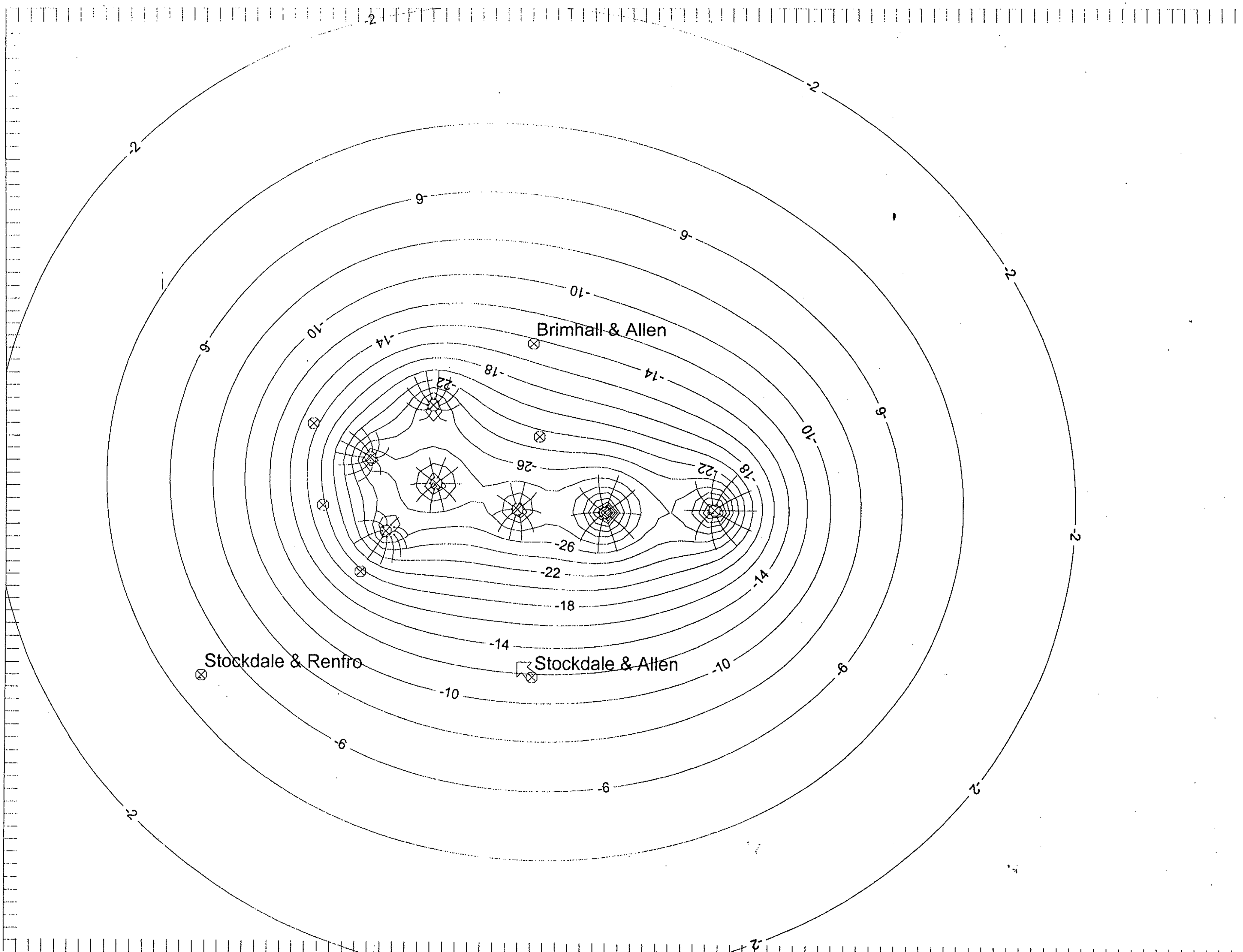
Set No. 3.

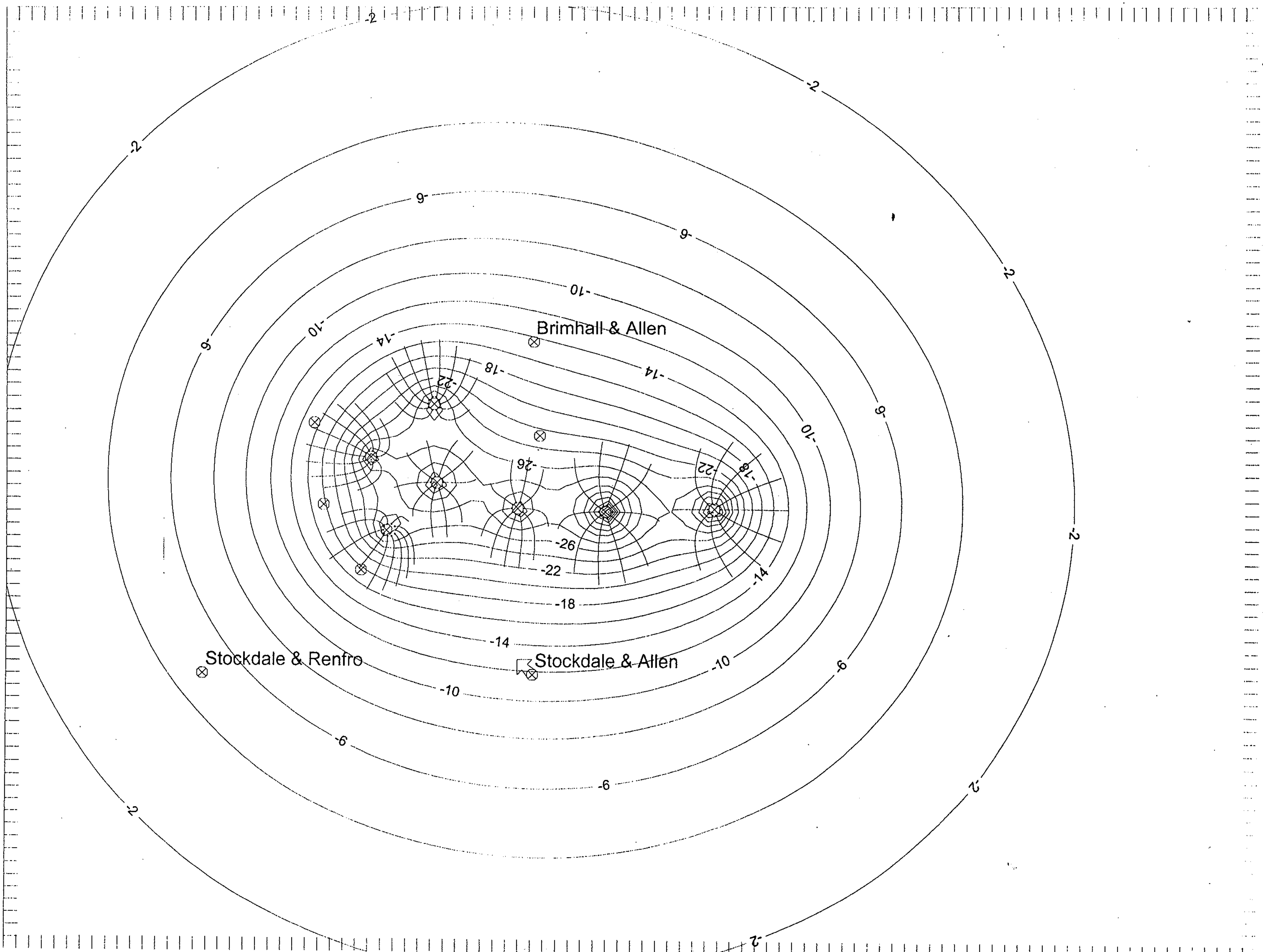
Note: only CHANGES are tabulated between Runs, from left to right.
Note: once made, CHANGES persist until modifed again.

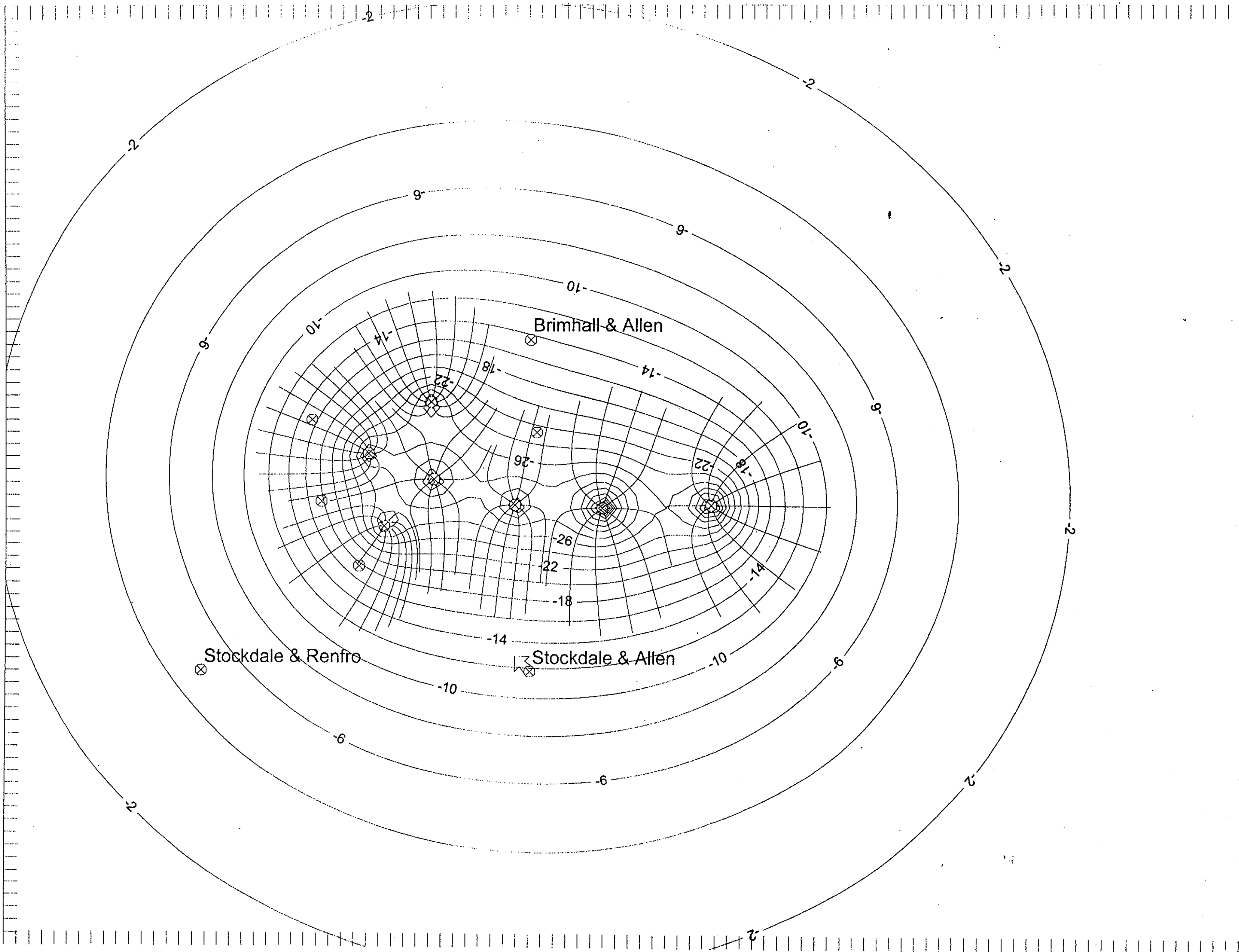
		base case												
model		leaky	leaky	leaky	leaky	leaky	leaky	leaky	leaky					
calc		transient	transient	transient	transient	transient	transient	transient	transient					
Run No.		ref.only	301	302	303	304	305							
H.Cond.	K	80												
Btm	B	-450												
Top	T	-200												
Ref Hd	R	0												
Grad	G	0												
Rchg	Rch	0												
Por	P	0.3												
Stor	S	5.60E-04												
Lkg	L	3200												
pump-t	t	300	10	30	100	300	770							
Ref Hd	Ref	0												
Grad	G	0												
Azim	A	135												
Ref-X	X	0												
Ref-Y	Y	0												
Const Hd	CH	no												
trace-t	tt	300												
Flow Rate	Q	full												
Wells	W	all												


Brimhall & Allen
Stockdale & Allen
Stockdale & Renfro
-2
-6
-10
-14
-18
-22
-26


Brimhall & Allen
Stockdale & Allen
Stockdale & Renfro
-2
-6
-10
-14
-18
-22
-26


Brimhall & Allen
Stockdale & Allen
Stockdale & Renfro
-2
-6
-10
-14
-18
-22
-26


Brimhall & Allen
Stockdale & Renfro
Stockdale & Allen
-2
-6
-10
-14
-18
-22
-26


Brimhall & Allen
Stockdale & Renfro
Stockdale & Allen
-2
-6
-10
-14
-18
-22
-26

Appendix 6.

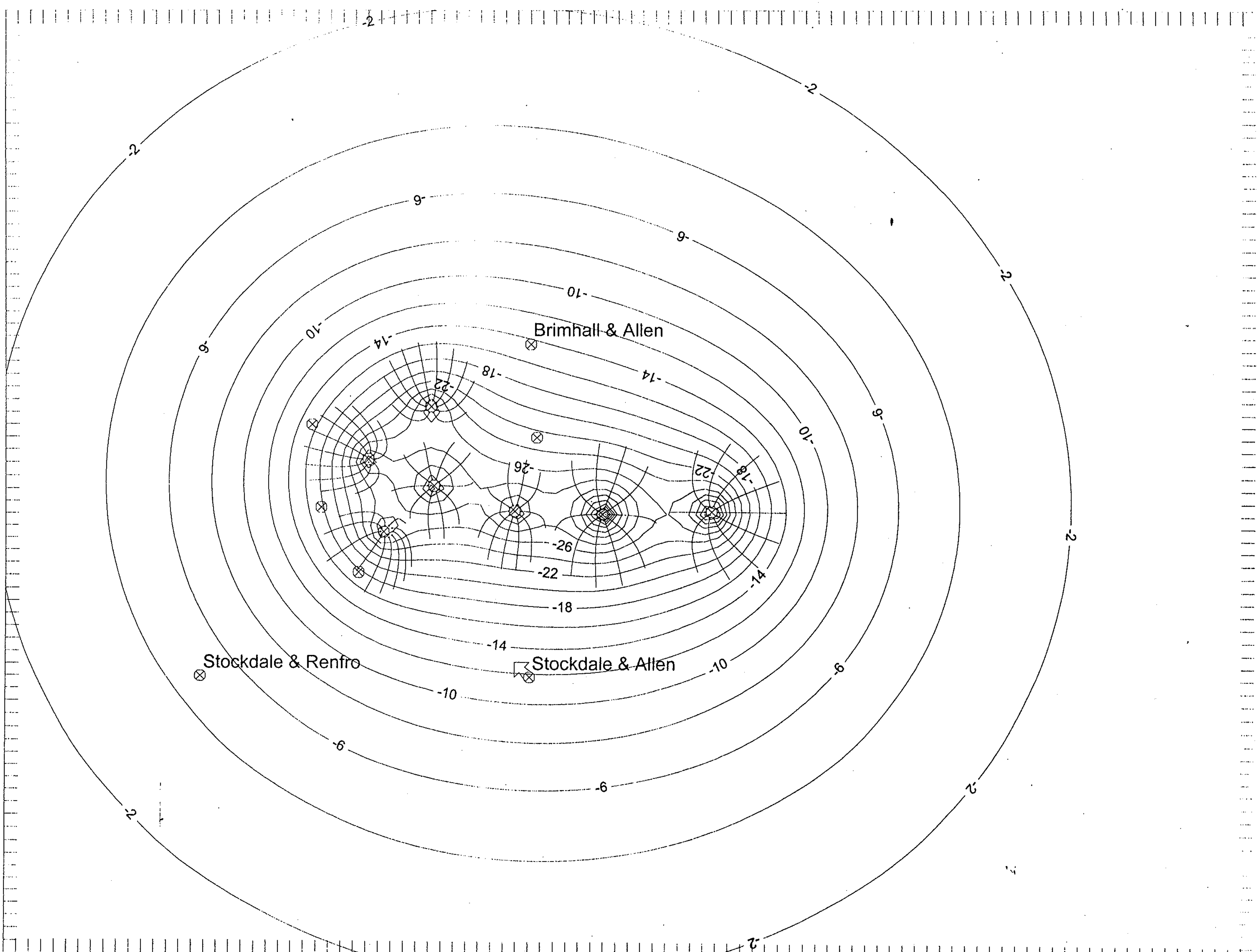
SET 04.

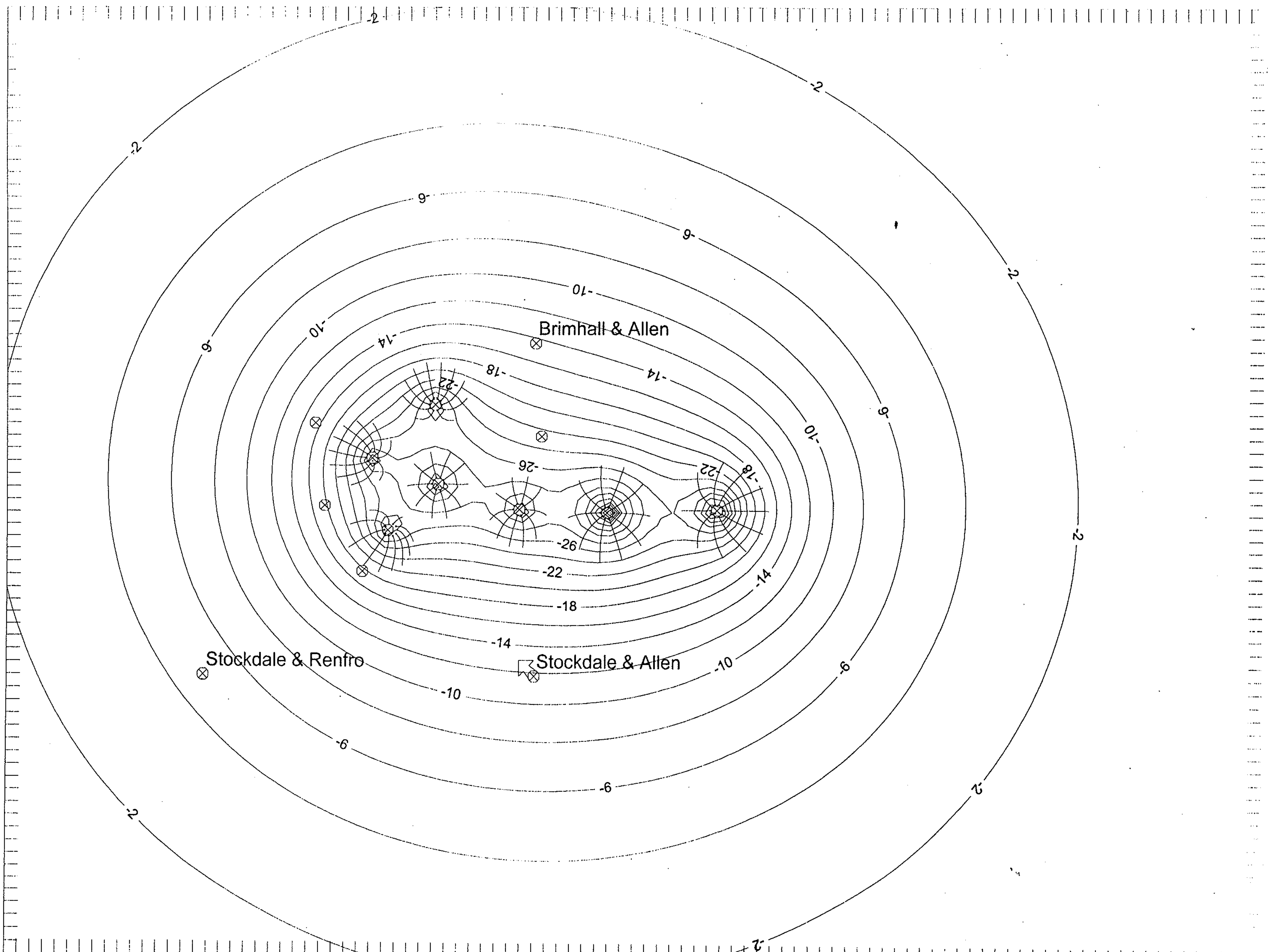
ID4 / KT / RRB Well Field Model Parameters.

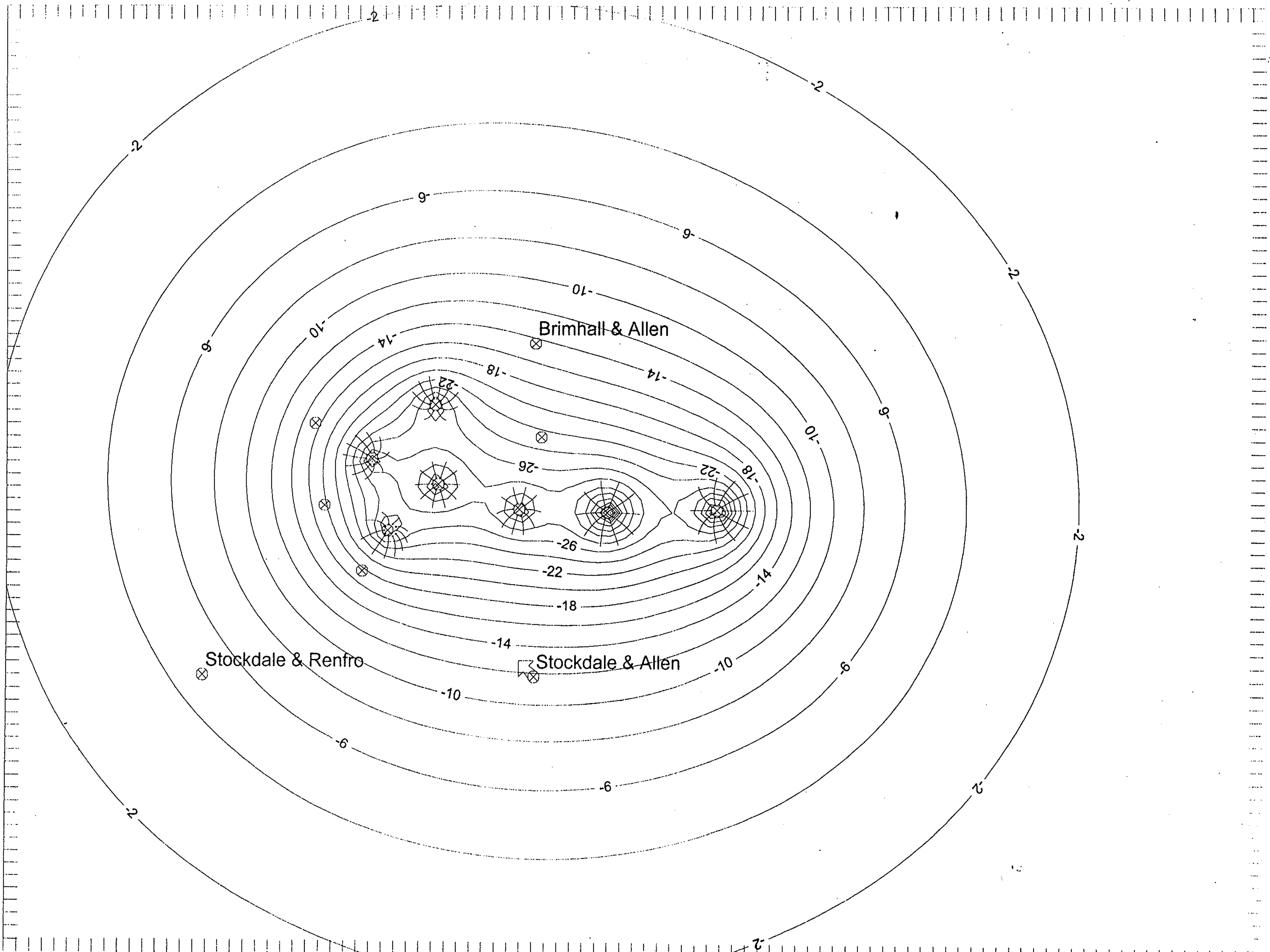
Set No. 456.

Note: only CHANGES are tabulated between Runs, from left to right.
Note: once made, CHANGES persist until modifed again.

		base case												
model		leaky	leaky	leaky	leaky		leaky	leaky	leaky		leaky	leaky	leaky	leaky
calc		transient	transient	transient	transient		transient	transient	transient		transient	transient	transient	transient
Run No.		ref.only	401	402	403		501	502	503		601	602	603	604
H.Cond.	K	80												
Btm	B	-450												
Top	T	-200												
Ref Hd	R	0												
Grad	G	0												
Rchg	Rch	0												
Por	P	0.3												
Stor	S	5.60E-04												
Lkg	L	3200												
pump-t	t	300	300	150	75		300							
Ref Hd	Ref	0												
Grad	G	0												
Azim	A	135												
Ref-X	X	0												
Ref-Y	Y	0												
Const Hd	CH	no												
trace-t	tt	300												
Flow Rate	Q	full					full	half	quarter		full			
Wells	W	all									all	A1-A5	ID4 1&2	A1 & ID4-2

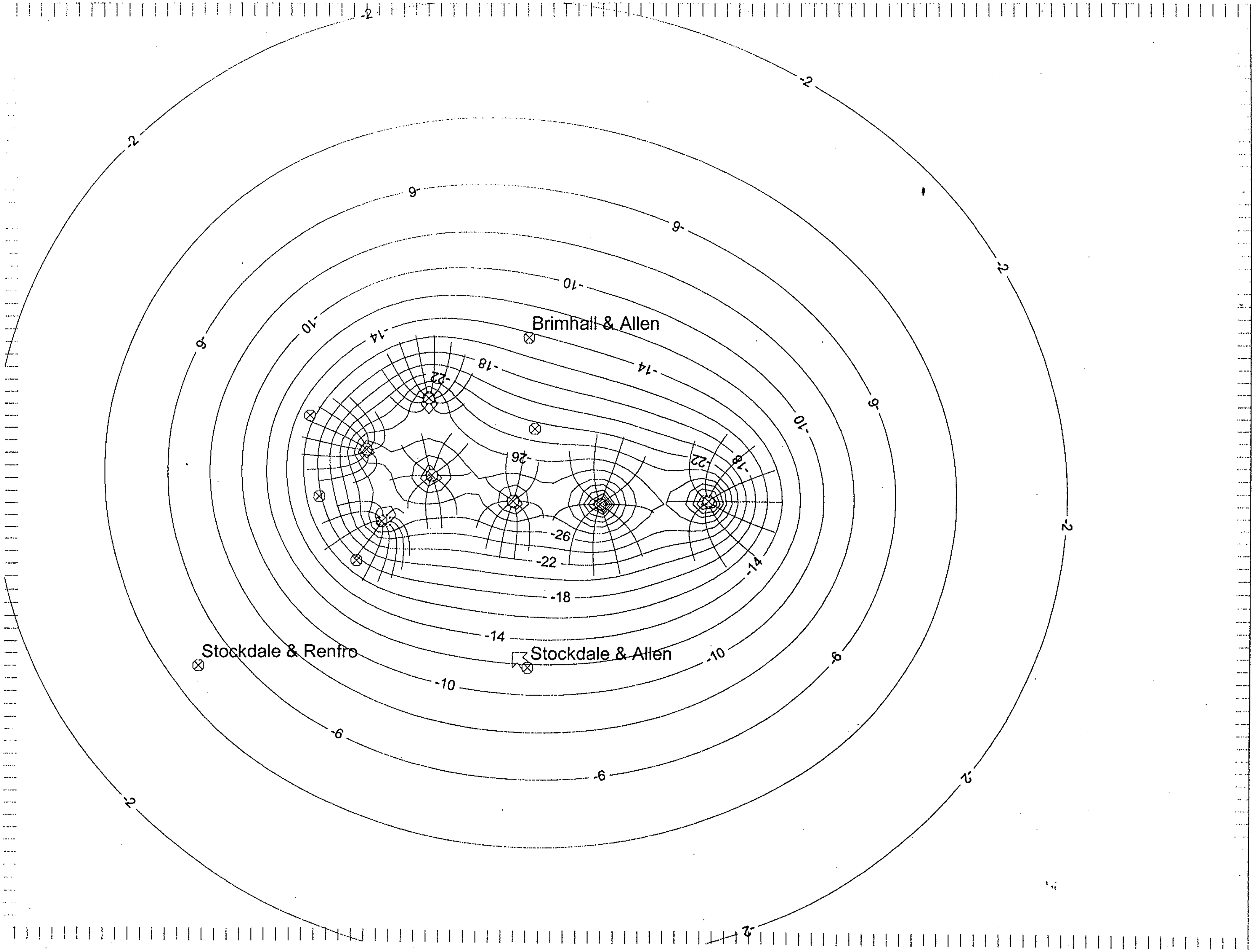

Brimhall & Allen
Stockdale & Allen
Stockdale & Renfro
-2
-6
-10
-14
-18
-22
-26

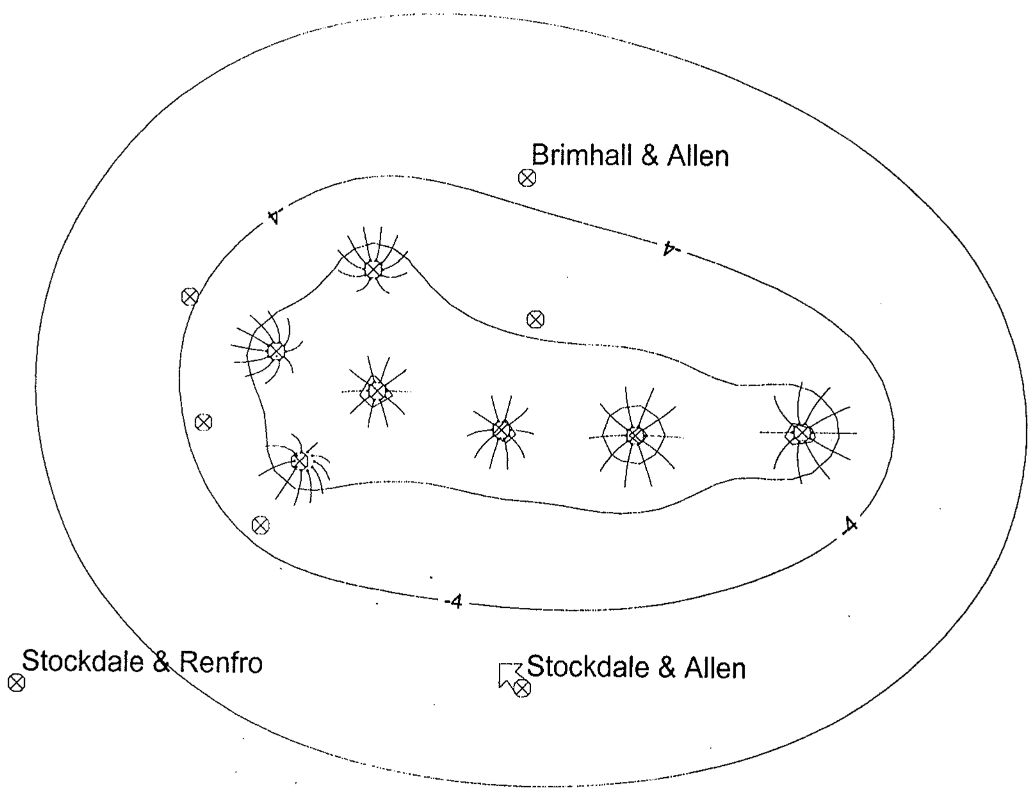

Brimhall & Allen
Stockdale & Renfro
Stockdale & Allen
-2
-6
-10
-14
-18
-22
-26


Brimhall & Allen
Stockdale & Allen
Stockdale & Renfro
-2
-6
-10
-14
-18
-22
-26

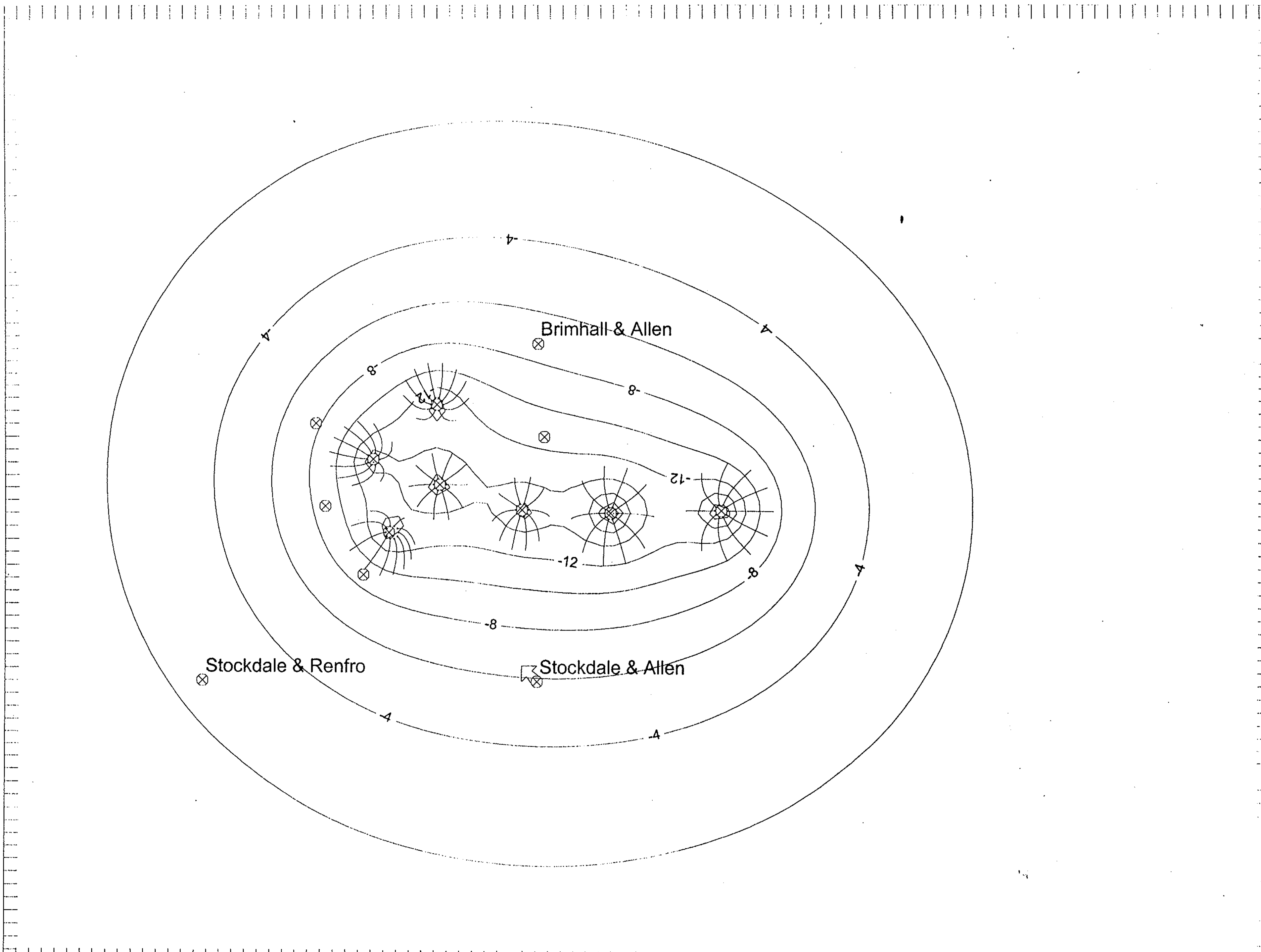
Appendix 6.

SET 05.


-2
-6
-10
-14
-18
-22
-26
Brimhall & Allen
Stockdale & Allen
Stockdale & Renfro



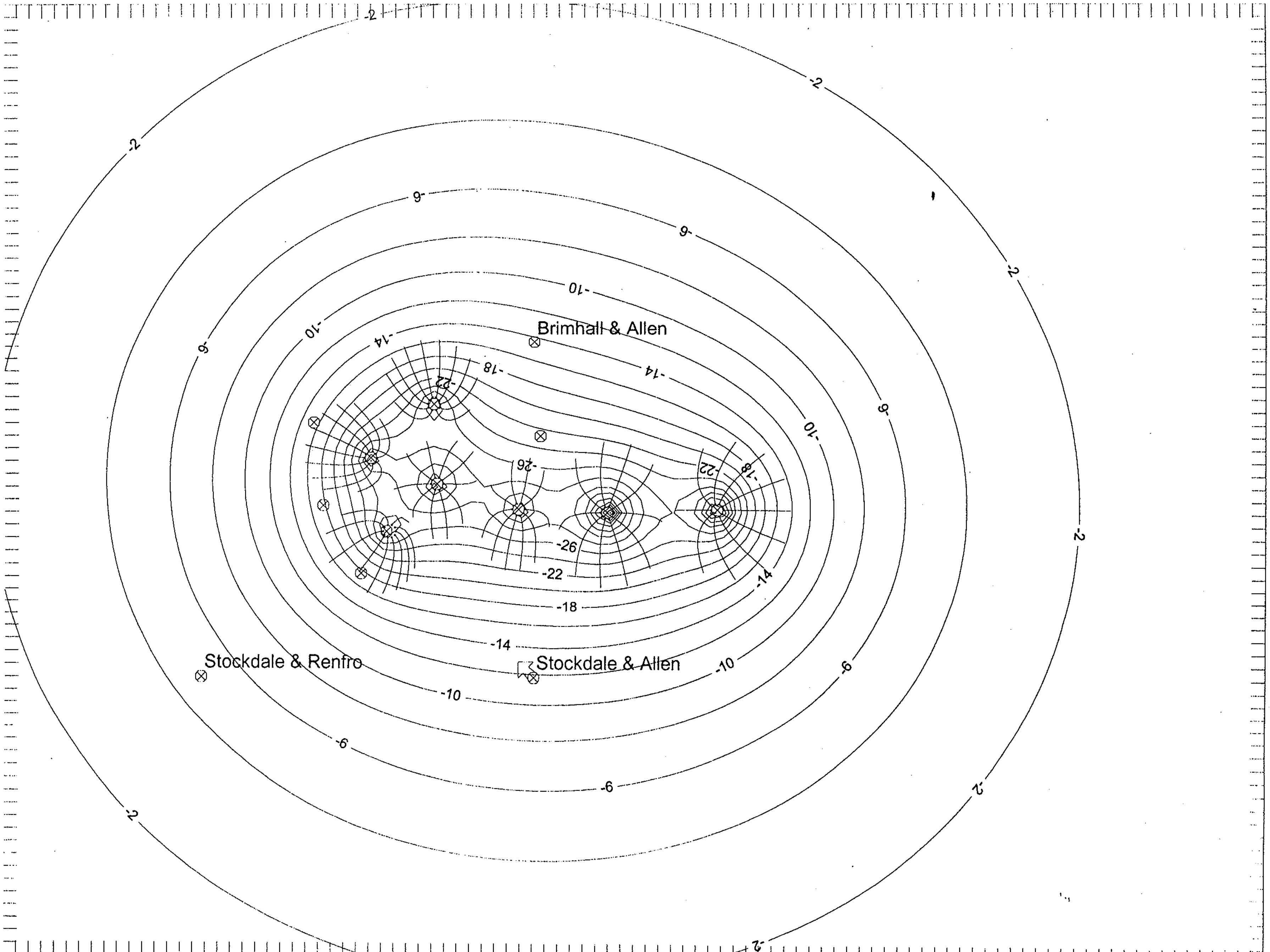
5/12



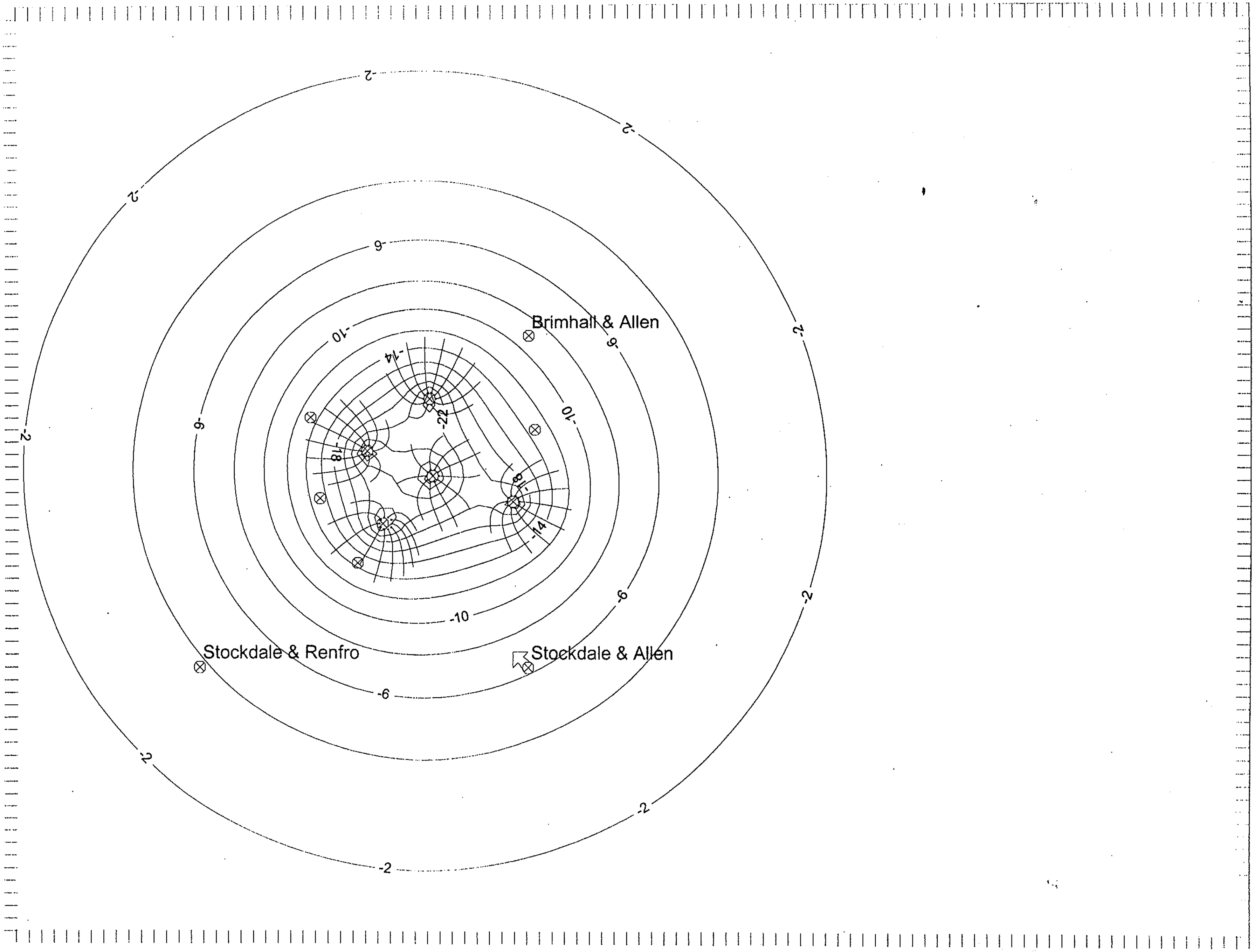
502

Appendix 6.

SET 06.

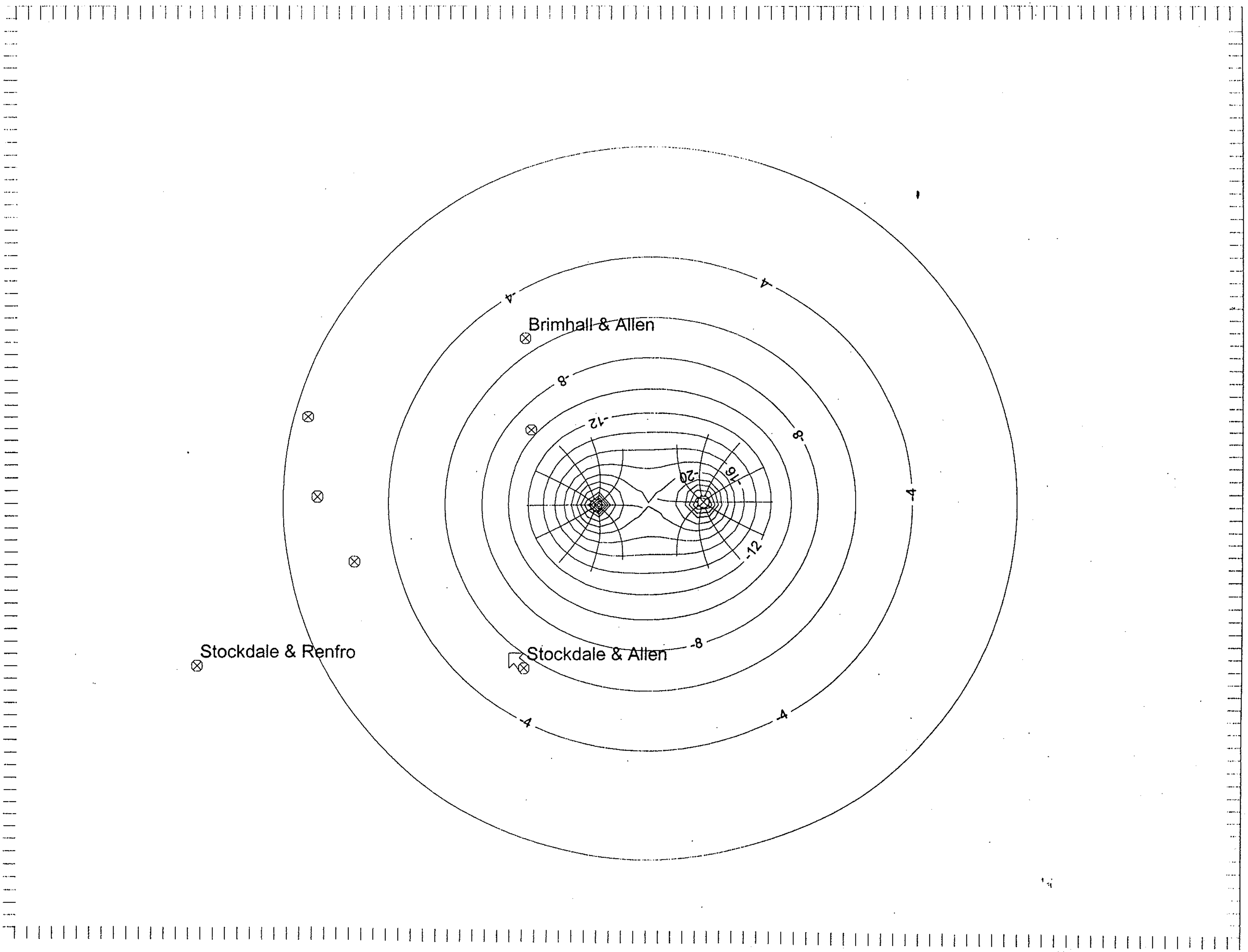


Brimhall & Allen
Stockdale & Allen
Stockdale & Renfro
-2
-6
-10
-14
-18
-22
-26

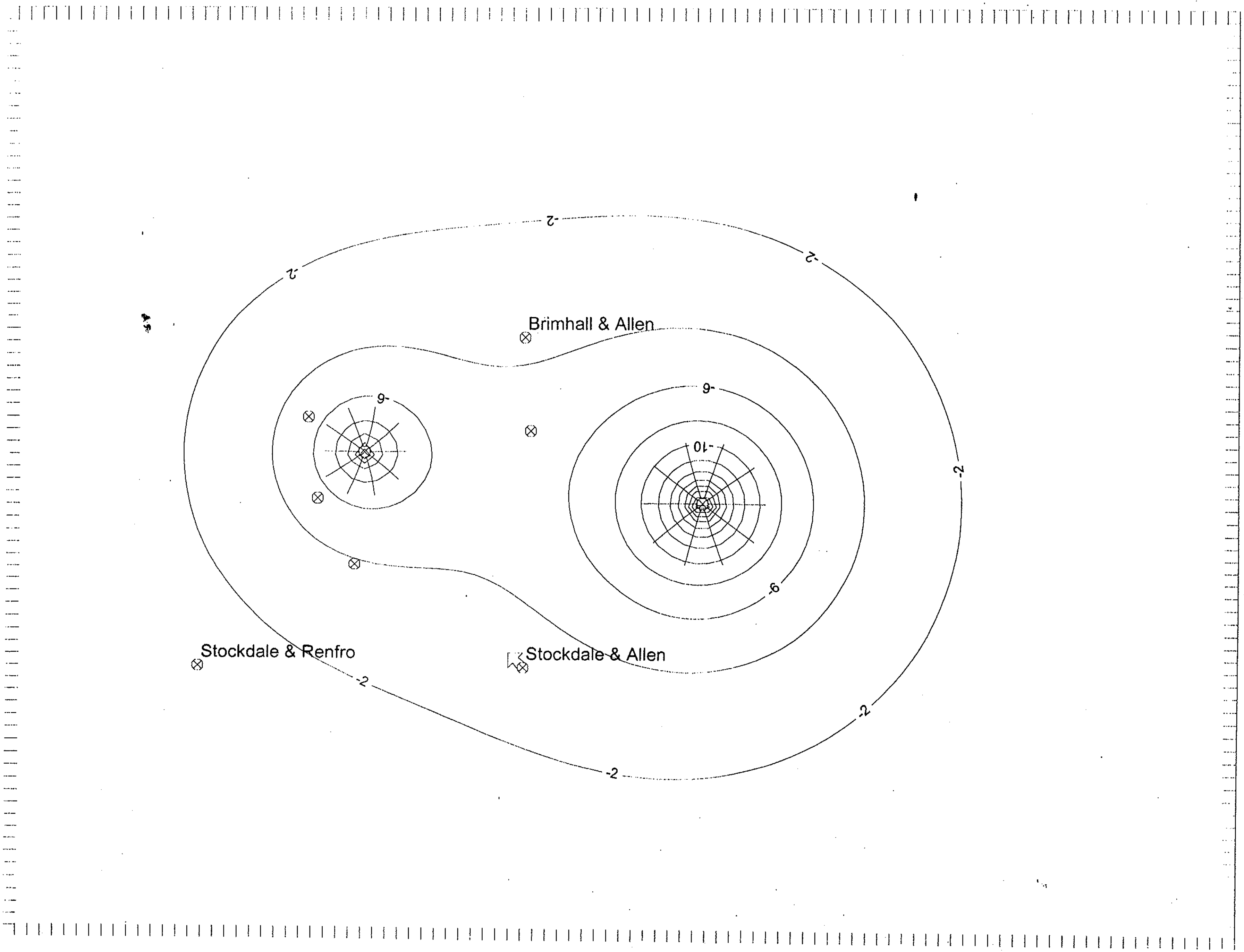


Brimhall & Allen
Stockdale & Renfro
Stockdale & Allen
-2
-6
-10
-14
-18
-22

(b)



-4
-4
-4
-4
Brimhall & Allen
-8
-8
-8
-12
-12
-16
-20
Stockdale & Renfro
Stockdale & Allen



604

Summary of Particle Trajectory and Capture Analyses, Sets 7 - 8.

The actual, physical volume of water which is captured by a single pumping well over any finite period of time does not come from the cone of depression surrounding the well but rather comes from the part of the aquifer enclosed by a vertical-sided circular cylinder centered on the well. The radius of this "capture" cylinder increases as the rate and duration of pumping increase and is easily determined arithmetically from the continuity equation. Except for the theoretical case of infinite pumping time in infinite aquifers, all capture zones have fully enclosing perimeters as seen on a map.

For a pair of pumping wells within each other's zone of influence, a drainage divide forms between them and either well can only drain water from its own side of the drainage divide. Therefore, the capture zones of these two wells do not overlap; they are ovals centered on each well and flattened against each other along the drainage divide. Theoretically, all water molecules on the divide are stagnant, not moving toward either well because they rest at a location of zero gradient where the opposing induced gradients exactly cancel each other out.

For a cluster of pumping wells, a system of drainage divides separates the capture zone of each well from that of every other well. The locations of the drainage divides depend on the relative pumping rates of the wells and the size and orientation of the natural gradient, all other things being equal. For base case pumping of all seven project wells, the drainage divide between the RRB and ID4 wells happens to nearly coincide with Allen Rd.

For a single well or cluster of wells in the presence of a groundwater gradient, the shape of this capture zone (CZ) is distorted into an oval which extends farther upgradient than downgradient. If a well or wells are pumped continuously, then this capture zone expands outward as the pumping time increases, but only out to a theoretical maximum limit in every direction. For a hypothetical infinite pumping time, the CZ is shaped like a parabola which is bounded on the downgradient end and transverse sides, and opens indefinitely in the upgradient direction. This particular, theoretical capture zone is the largest possible capture zone for a given set of parameters and we define this theoretical perimeter as the capture zone limit (CZL). For all *finite* pumping times, the actual capture zone is smaller than, inside of, and conformal to the CZL.

In reality, i.e., in the real aquifer and for real pumping operations in this project area, a real capture zone may approach the location of the CZL in downgradient and transverse locations in just a few years of pumping but will never extend farther upgradient than the location of the recharge boundary caused by the Kern River Channel. The real capture zone will stop expanding as soon as the total recovery rate is fully balanced by aquifer recharge at any or all of the aquifer boundaries.

Any water within the CZL has the potential to be ultimately recovered by the wells if pumping continues long enough. Any water outside of the CZL will never be captured by the wells as long as conditions remain the same. Our forecast of the location of the CZL is predicated on constant conditions and our choice of conditions is based on our guess of the long term average ground water gradient behavior in the project area. The most significant possible exception is based on the recognized swing in the direction of the prevailing northwesterly natural gradient to westerly when climatic conditions change from wet to dry.

On SSS's drawdown maps, the spider- like particle trajectories centered on each well extend outward to the most distant location from which water is captured in a user- specified period of time. The curved shapes and convergence of some trajectories represent the effect of the non-Darcy flow regime within the capture zone. The non- Darcy flow also causes water molecules to speed up as they follow a particle trajectory inward toward a well. Although we chose to not show velocities or interval transit times on the maps, the qualitative measure of speed increase is the rate of convergence of the particle trajectories as seen on the maps.

The inward radial gradient which develops within the cone of depression surrounding a pumping well is not the only driving force which moves water toward the well. The movement of groundwater along any particle trajectory is always due to the combined total influence of all sources, even the influences of other wells. Particle trajectories and capture zones have only limited relevance unless they are superimposed on a realistic approximation of the local ground water gradient for the project area, including any other predictable influences.

The natural groundwater gradient is of particular importance. In the presence *and* absence of pumping, upgradient water continues to move toward a well due to the presence of the natural groundwater gradient. Therefore, a plume or slug of groundwater contamination which is upgradient of a well will move toward the well due to the natural gradient whether the well is pumping or not. The primary lesson to be appreciated is that if and when contamination shows up in a well which had previously been contaminant- free, it should not be assumed that pumping caused the contamination to show up until it is shown that it wasn't headed there anyway under the natural gradient. The presumption of causality is a common pitfall which is difficult to avoid without a careful analysis of particle trajectory and capture analysis.

The 2003 KDSA report presents two different groundwater conditions in their impact analysis of the project area. One condition represents the *northwesterly* water table gradient which typifies wet years when Kern River recharge occurs and the orientation and close proximity of the recharge mound dominates the local hydrology. The second condition represents the *westerly* water table gradient which typifies dry years when river recharge does not occur and more distant features of the aquifer to the east dominate the local hydrology. We have not independently verified the persistence or representativeness of these two specific scenarios within this scope of work, but we have reviewed the KCWA ground water elevation maps for the years since 1990 and these two scenarios appear to be consistent with the observed overall trends.

For our scope of work, we have adopted three hypothetical groundwater gradient scenarios; one in which a NW gradient of -0.002 points northwesterly (135° left azimuth from east), a second in which a W gradient of -0.002 points westerly (180° left azimuth from east), and a third set in which we model active recharge in the Kern River Channel superimposed on a WNW gradient of -0.001 which points west-northwesterly (180°, 150°, and135° left azimuth from east). In all scenarios, we have assumed that the ground water at the beginning is 100 ft deep at the intersection of Stockdale Hwy and Allen Rd, which is approximately correct for Summer, 2004, and that the gradient defines the depth to water elsewhere relative to this reference point.

<u>Set 7. Base case superimposed on a natural GW gradient (t = 300, 770d).</u>

For the base case superimposed on the NW gradient, the 770- day capture zone extends a maximum of about 2,000 ft directly upgradient and a maximum of about 1,500 ft downgradient from the wells at opposite ends of the well array. Ignoring the fact that this is non-darcy, variable velocity flow, the average groundwater particle velocities moving from the capture perimeter to the wells during the 770 day capture period range from 2.1 - 2.7 ft/d; slower at first and faster just prior to capture. For comparison purposes, the calculated Darcy groundwater flow velocity under the uniform natural gradient alone is about 0.46 ft/d for base case parameters.

For the base case superimposed on the W gradient, the dimensions and flow velocities are essentially the same as the NW gradient case except that the orientation is slightly different.

For the base case superimposed on either gradient, the 300- day capture zone extends a maximum of about 1,300 ft directly upgradient and a maximum of about 1,000 ft downgradient from the wells at opposite ends of the well array. Ignoring the non-darcy flow factor, the average groundwater particle velocities moving from the capture perimeter to the wells during the 300 day capture period range from 3.3 - 4.3 ft/d. For comparison purposes, the calculated Darcy groundwater flow velocity under the uniform natural gradient alone is about 0.46 ft/d for base case parameters.

<u>Set 8. Base case superimposed on a natural GW gradient (t = 1, 3, 5, 10, & 20 yr).</u>

These cases are the same as the previous ones except that the hypothetical capture zones extend farther upgradient for much longer time pumping periods . This is not completely realistic since the assumption that the aquifer is laterally infinite does not actually apply to the project area, but it gives a general impression of what upgradient areas are- or are not- within the capture zone for the specified durations of continuous pumping. Since the groundwater flow velocities are less under the natural gradient than under pumping, these scenarios represent maximum capture zones for the given time periods, subject to the representativeness of the assumptions and parameters. For intermittent pumping over these time periods, the actual capture zone will be smaller than those presented in the following cases.

Based on these models, the farthest upgradient extents of the capture zone are 1,400, 2,100, 3,600, 5,200, and 5,900 ft for continuous pumping times of t = 1, 3, 5, 10, & 20 yr, respectively. These scenarios show which areas surrounding the project are inside and outside the capture zone for the specified pumping times. Groundwater which is outside these capture zones will not be captured by any scenario of less-than-continuous pumping, all else being equal, over these respective time durations.

If contamination shows up in one or more wells during actual project pumping, then this analysis limits the possible directions from which it may have come. The duration of pumping up to the onset of contamination can be used to determine how far away the contamination was before it showed up at the wells.

Conversely, if a source of contamination or an existing plume is known to be upgradient from the project wells, then the obvious question is, "How long will the contamination take to get here?". The upgradient limits of the capture zones for the specified times give ballpark estimates, but this oversimplified analysis ignores other issues of contaminant transport which bear on this determination. For example, longitudinal dispersion causes the leading edge of a plume to travel faster than the center of the plume so "breakthrough" arrives sooner than predicted. On the other hand, many contaminants have flow velocities which are slower than the groundwater flow velocity, so this retardation delays the arrival. SSS has performed such an analysis for several types of contaminant plumes, but a more detailed discussion here is outside the scope of work. The point is that other factors determine the motion of contaminants in groundwater and so it is not appropriate to assume that contaminants move at the same speed as the groundwater that they are dissolved in.

Appendix 6.

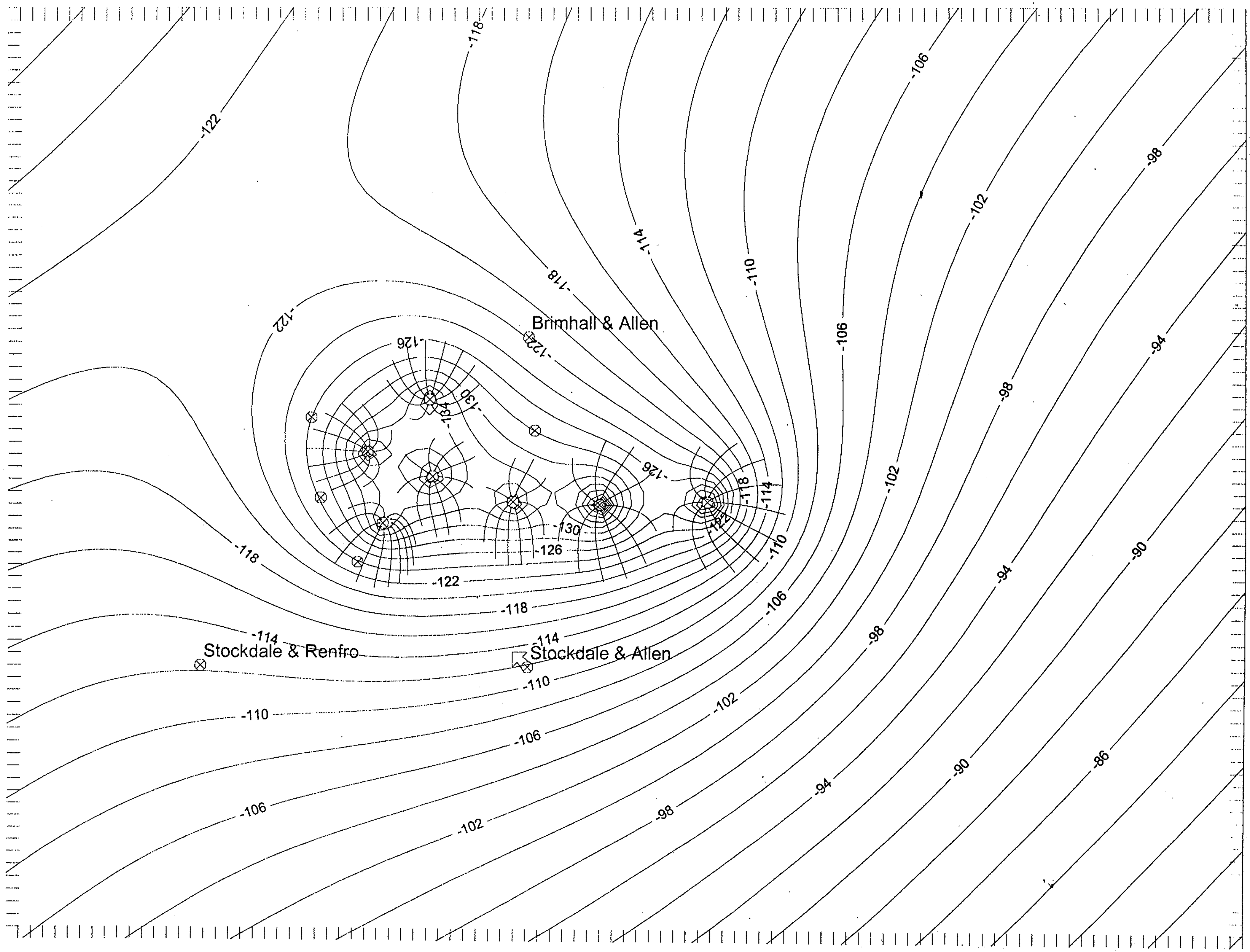
SET 07.

ID4 / KT / RRB Well Field Model Parameters.

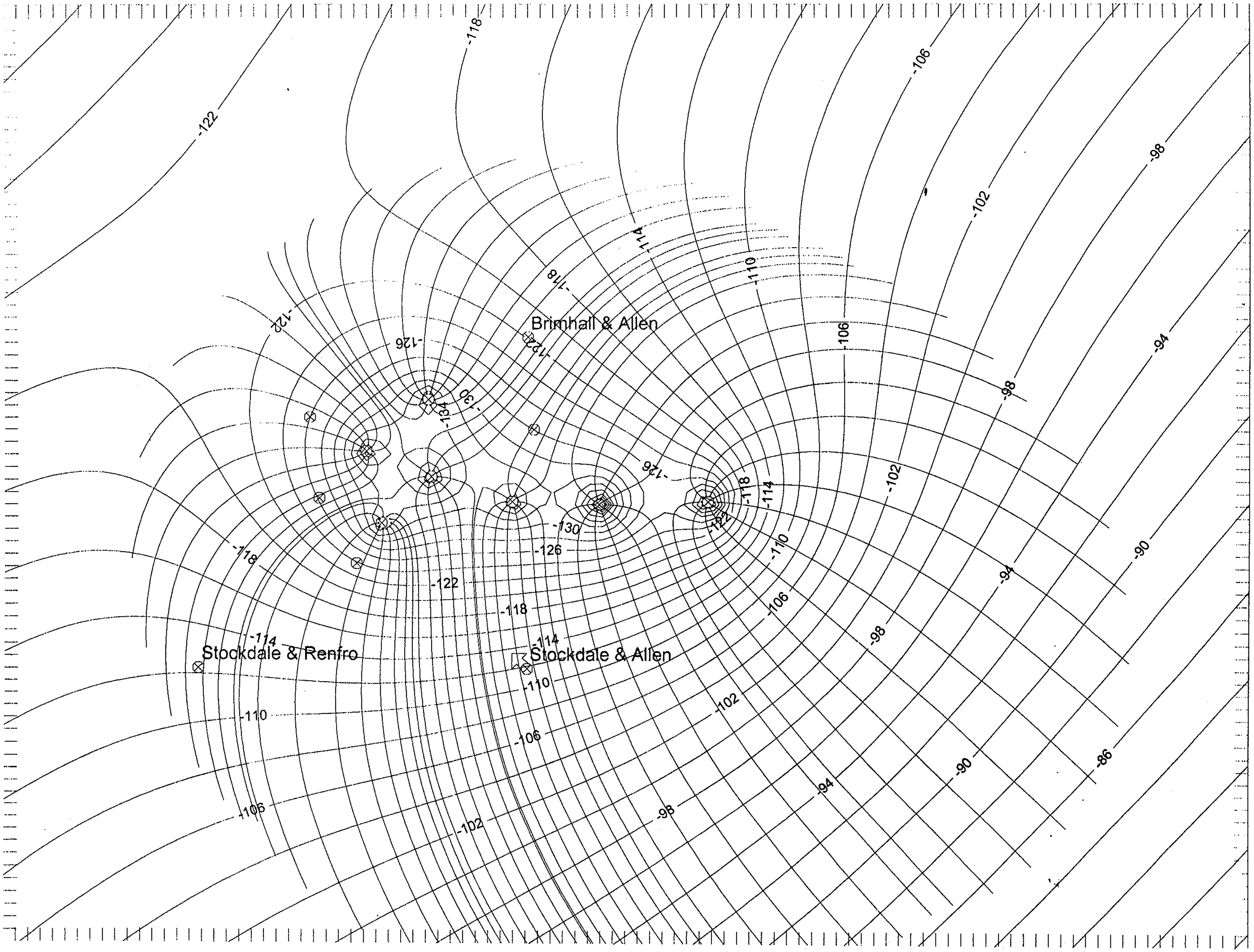
Set No. 7.

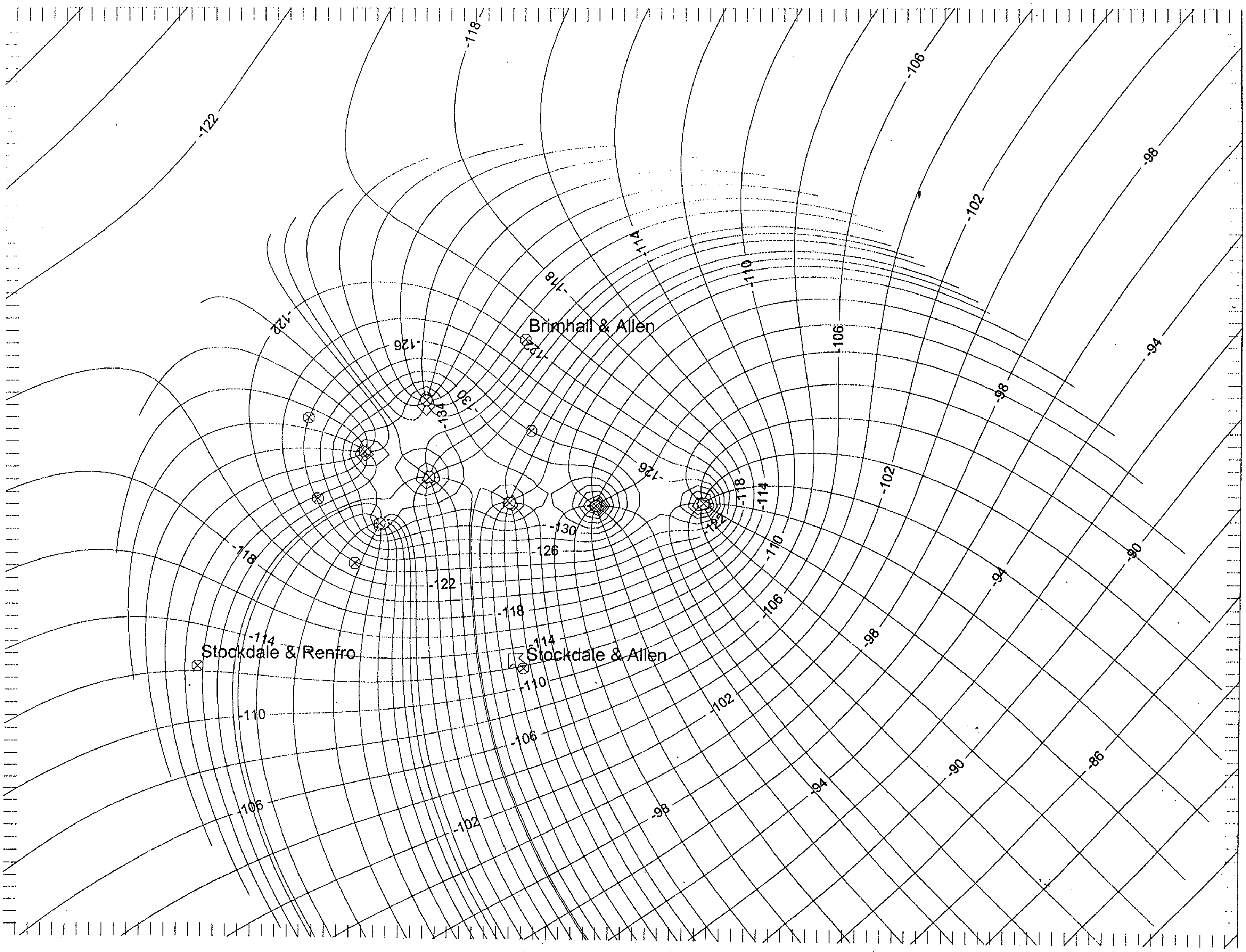
Note: only CHANGES are tabulated between Runs, from left to right.
Note: once made, CHANGES persist until modifed again.

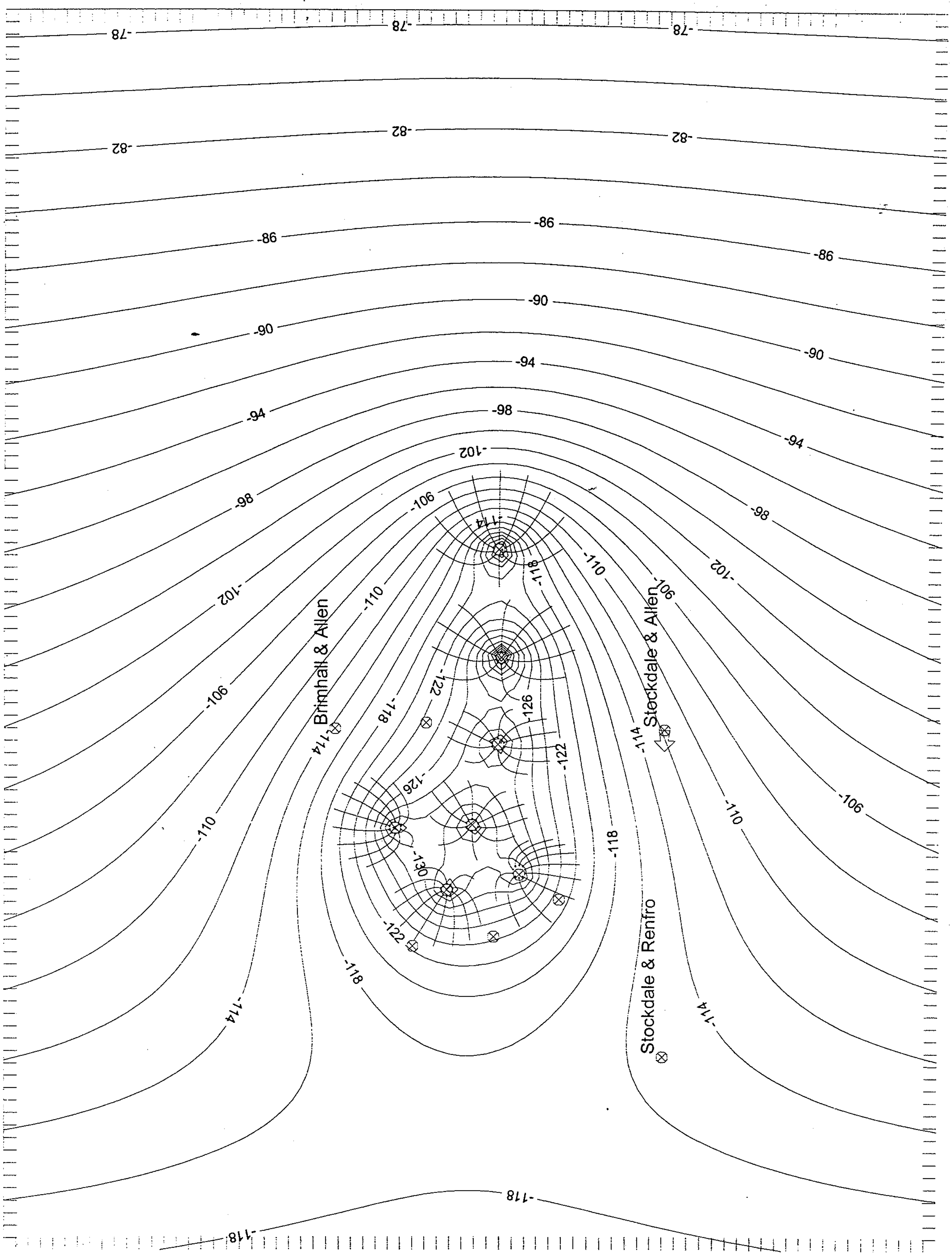
		base case												
model		leaky	leaky	leaky	leaky	leaky	leaky	leaky						
calc		transient	transient	transient	transient	transient	transient	transient						
Run No.		ref.only	701	702	703	704	705	706						
H.Cond.	K	80												
Btm	B	-450												
Top	T	-200												
Ref Hd	R	0	-100											
Grad	G	0	0.002											
Rchg	Rch	0												
Por	P	0.3												
Stor	S	5.60E-04												
Lkg	L	3200												
pump-t	t	300		7305	10958	300	7305	10958						
Ref Hd	Ref	0	-100											
Grad	G	0	0.002											
Azim	A	135				180								
Ref-X	X	0												
Ref-Y	Y	0												
Const Hd	CH	no												
trace-t	tt	300		7305	10958	300	7305	10958						
Flow Rate	Q	full												
Wells	W	all												

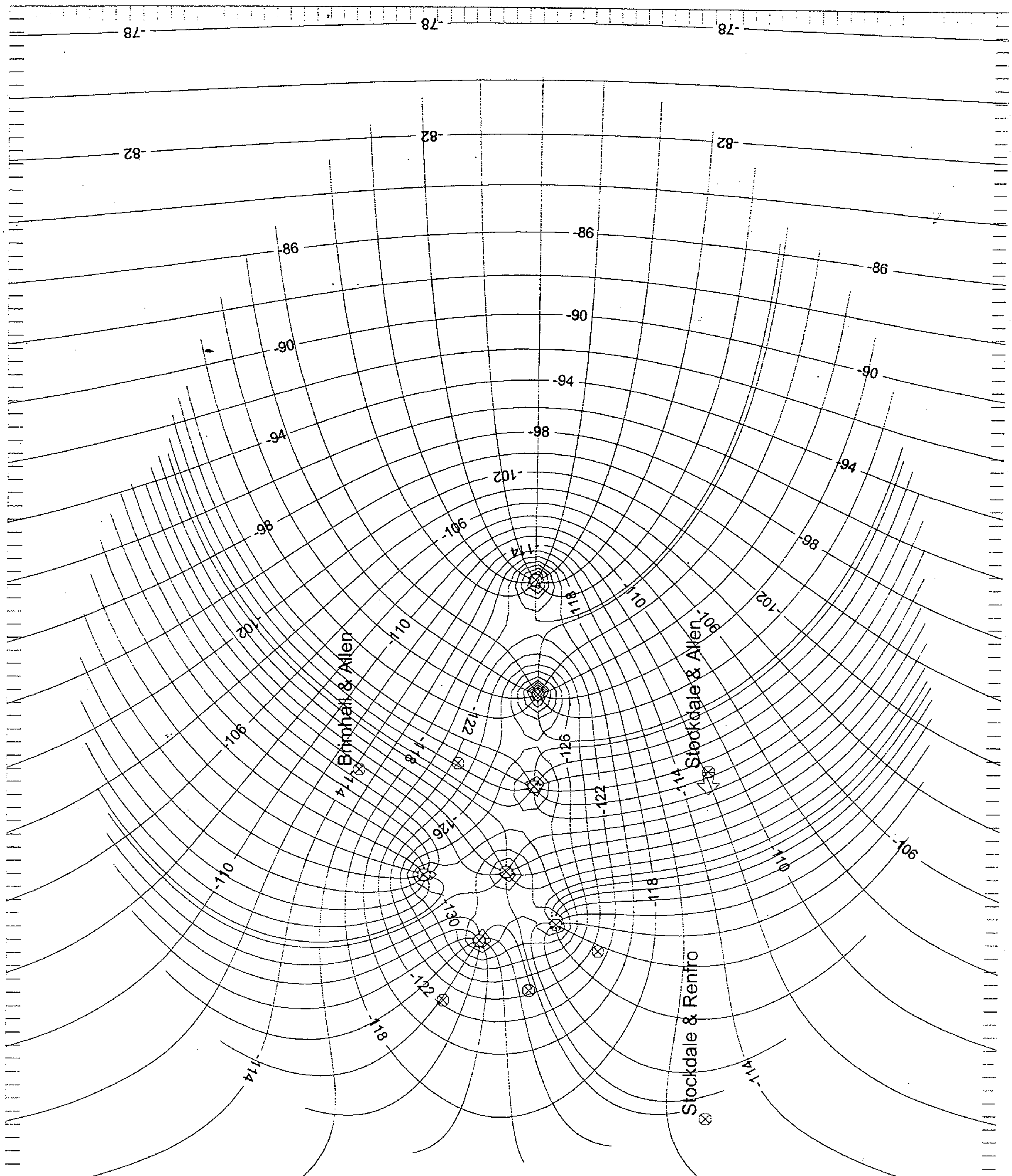


Brimhall & Allen
Stockdale & Allen
Stockdale & Renfro
-86
-90
-94
-98
-102
-106
-110
-114
-118
-122
-126
-130
-134

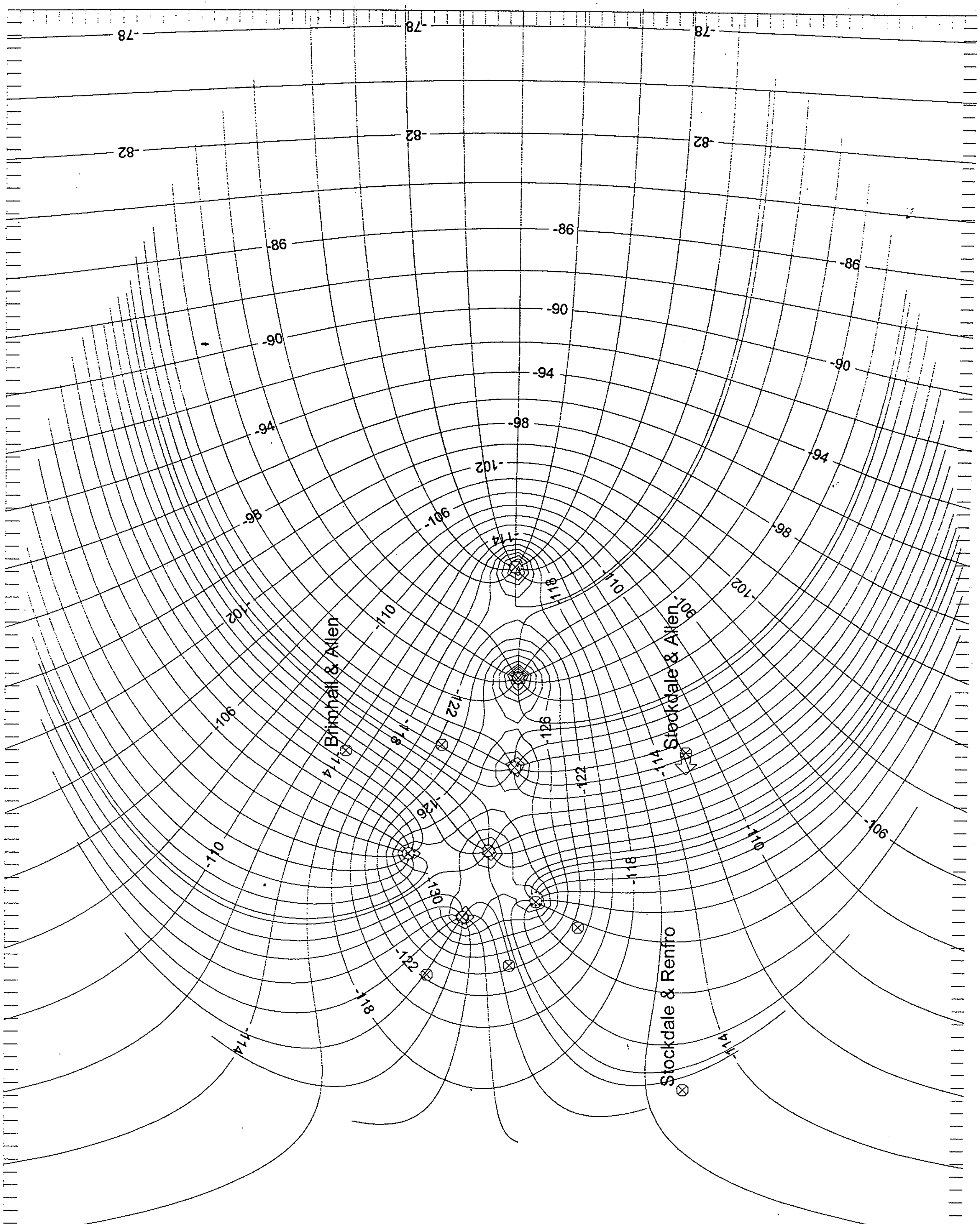

Brimhall & Allen
Stockdale & Allen
Stockdale & Renfro


Brimhall & Allen
Stockdale & Allen
Stockdale & Renfro
-86
-90
-94
-98
-102
-106
-110
-114
-118
-122
-126
-130
-134


-78
-82
-86
-90
-94
-98
-102
-106
-110
-114
-118
-122
-126
-130
Brimhall & Allen
Stockdale & Allen
Stockdale & Renfro


-78
-82
-86
-90
-94
-98
-102
-106
-110
-114
-118
-122
-126
-130
Brimhall & Allen
Stockdale & Allen
Stockdale & Renfro

705


Brimhall & Allen
Stockdale & Allen
Stockdale & Renfro
-78
-82
-86
-90
-94
-98
-102
-106
-110
-114
-118
-122
-126
-130

706

Appendix 6.

SET 08.

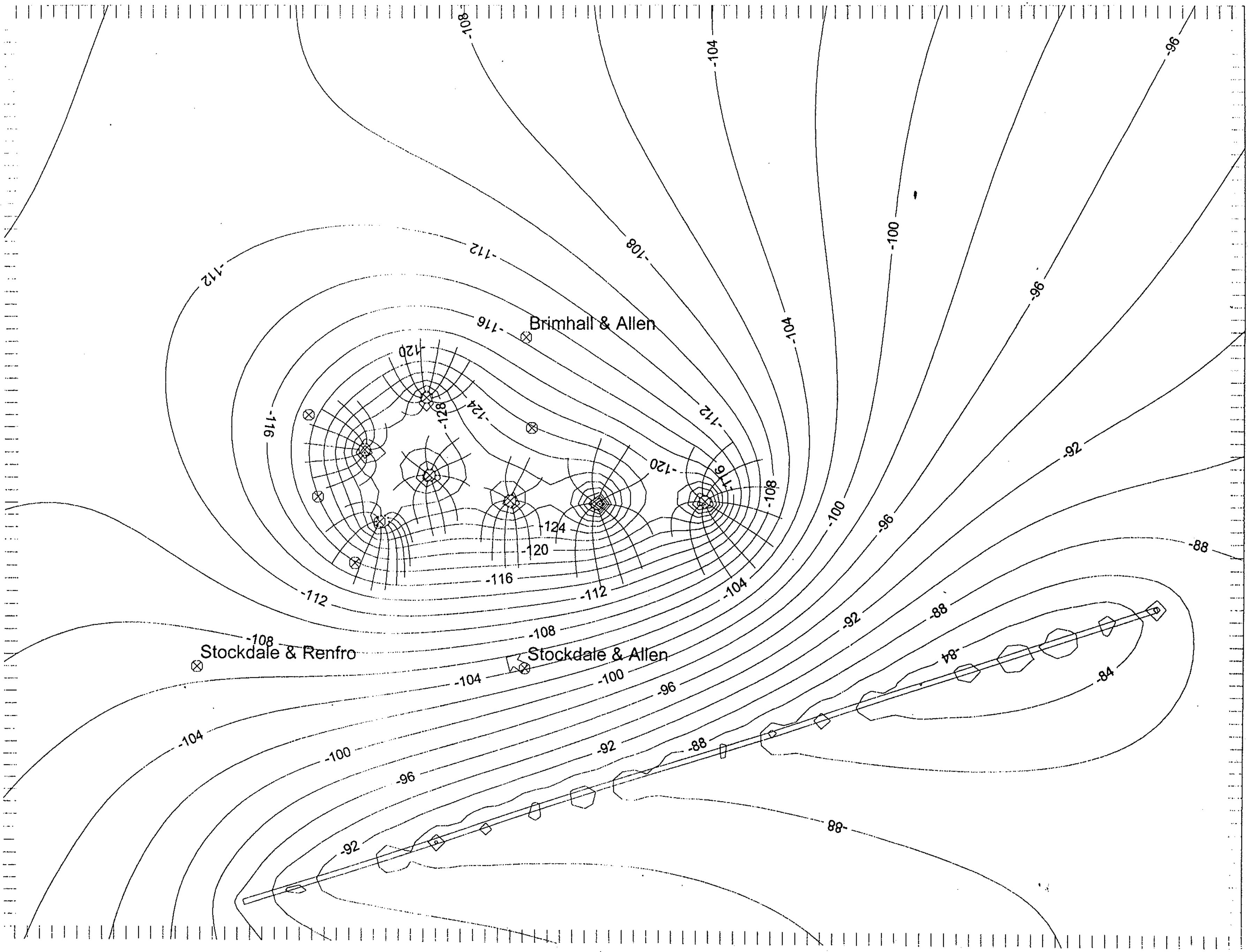
ID4 / KT / RRB Well Field Model Parameters.

Set No. 8.

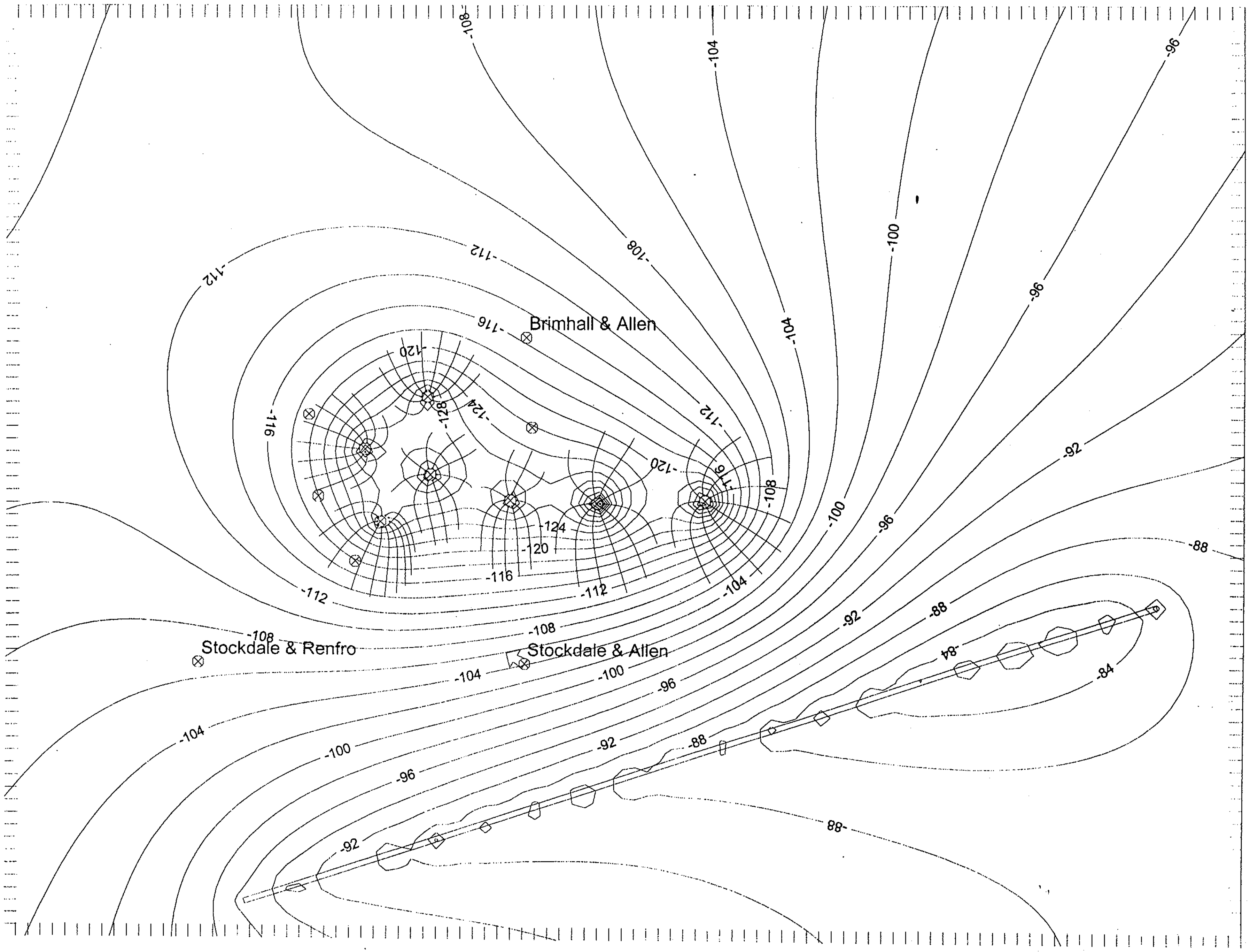
Note: only CHANGES are tabulated between Runs, from left to right.
Note: once made, CHANGES persist until modifed again.

		base case												
model		leaky	leaky	leaky	leaky	leaky	leaky	leaky	leaky	leaky				
calc		transient	transient	transient	transient	transient	transient	transient	transient	transient				
Run No.		ref.only	801	802	803	804	805	806	807	808				
H.Cond.	K	80												
Btm	B	-450												
Top	T	-200												
Ref Hd	R	0	-100											
Grad	G	0	0.001											
Rchg	Rch	0												
Por	P	0.3												
Stor	S	5.60E-04												
Lkg	L	3200												
pump-t	t	300		365	731	1096	1826	3653	7305	10958				
Ref Hd	Ref	0	-100											
Grad	G	0	0.001											
Azim	A	135	150											
Ref-X	X	0												
Ref-Y	Y	0												
Const Hd	CH	no												
trace-t	tt	300	300	365	731	1096	1826	3653	7305	10958				
				(1 yr)	(2 yr)	(3 yr)	(5 yr)	(10 yr)	(20 yr)	(30 yr)				
Flow Rate	Q	full												
Wells	W	all												
River Rch	Q	0	-210											

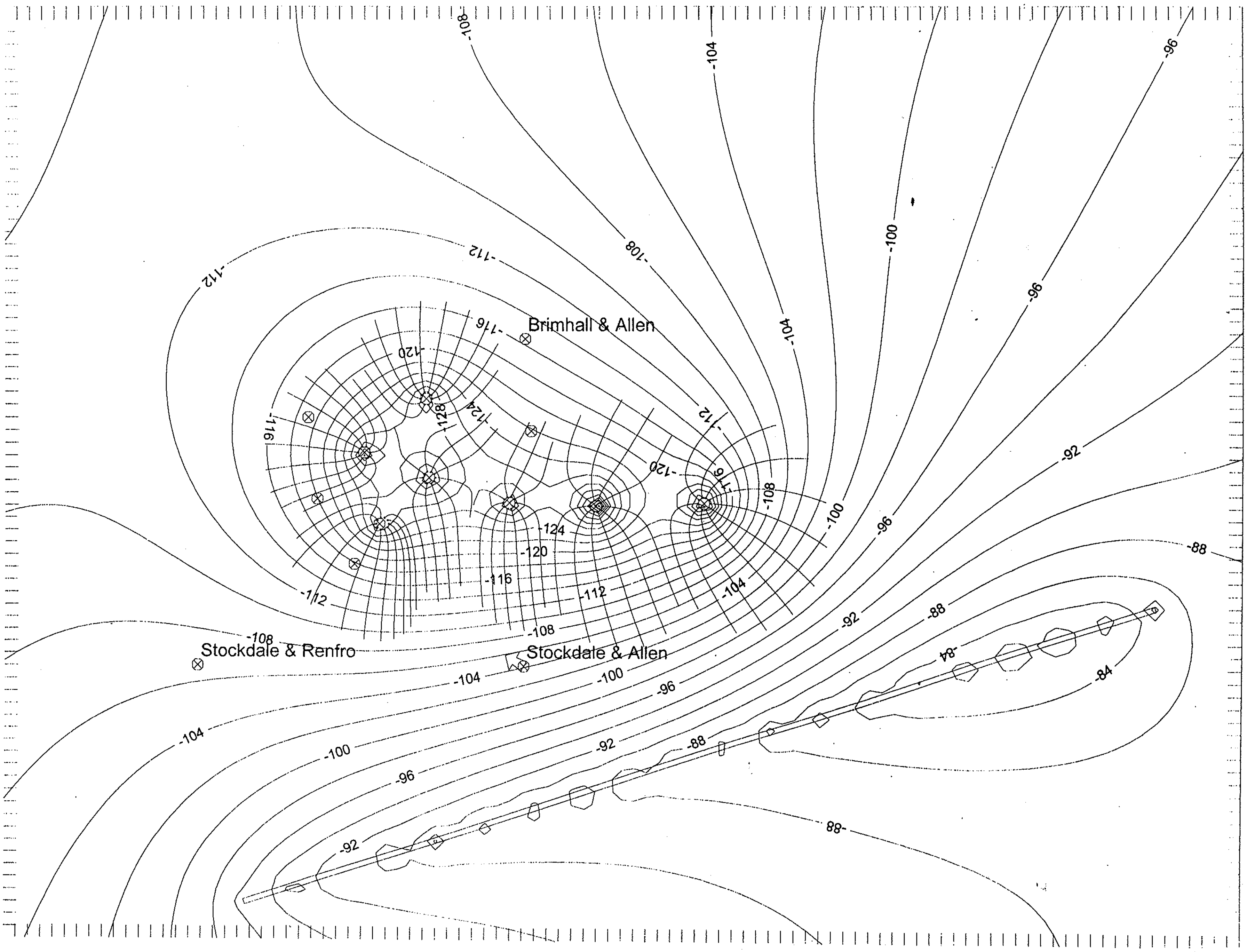
river coords: LL (-4500, -3400) UR (10200, 1000)


Brimhall & Allen
Stockdale & Renfro
Stockdale & Allen
-84
-88
-92
-96
-100
-104
-108
-112
-116
-120
-124
-128

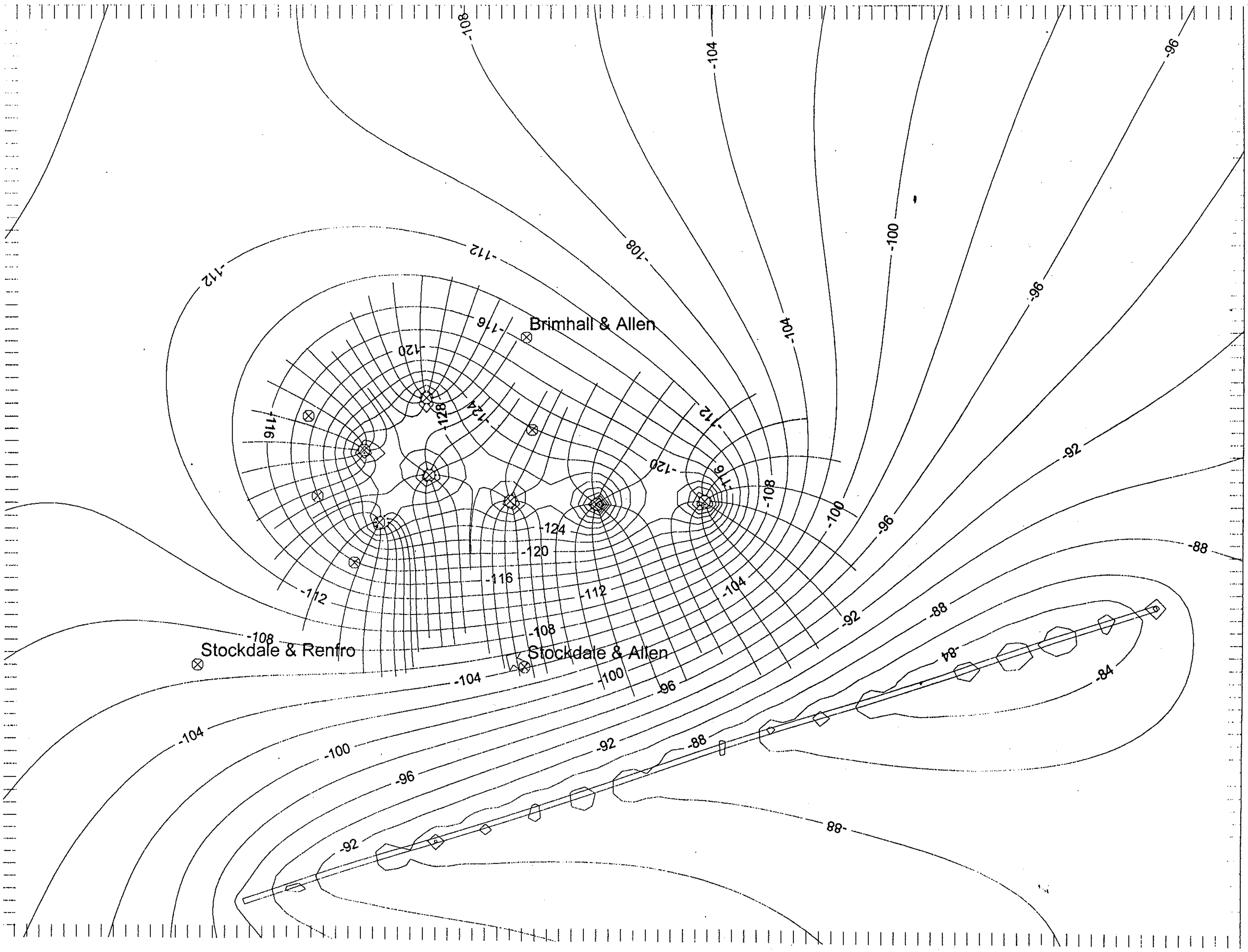
108


Brimhall & Allen
Stockdale & Allen
Stockdale & Renfro
-84
-88
-92
-96
-100
-104
-108
-112
-116
-120
-124
-128

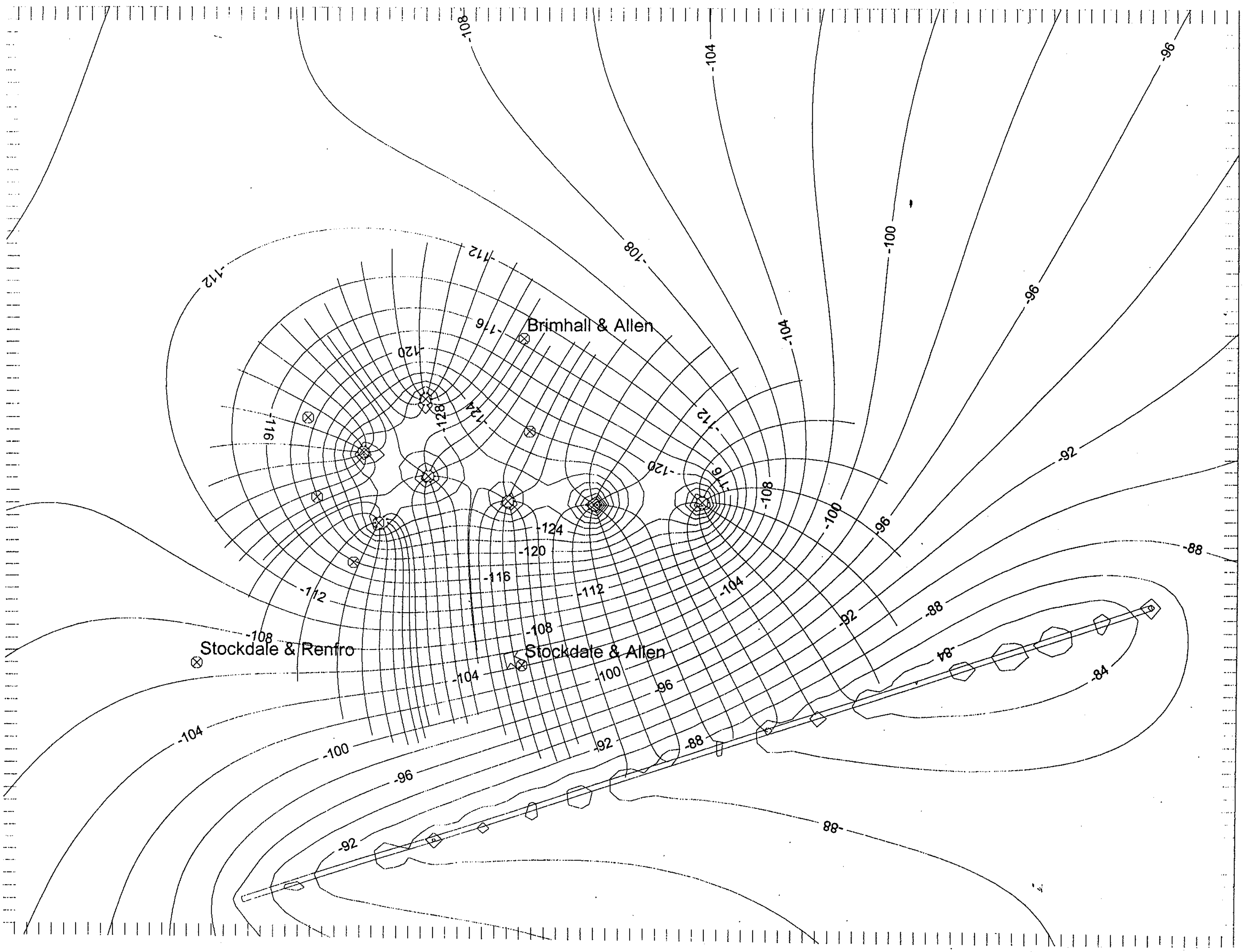
862


Brimhall & Allen
Stockdale & Allen
Stockdale & Renfro
-84
-88
-92
-96
-100
-104
-108
-112
-116
-120
-124
-128

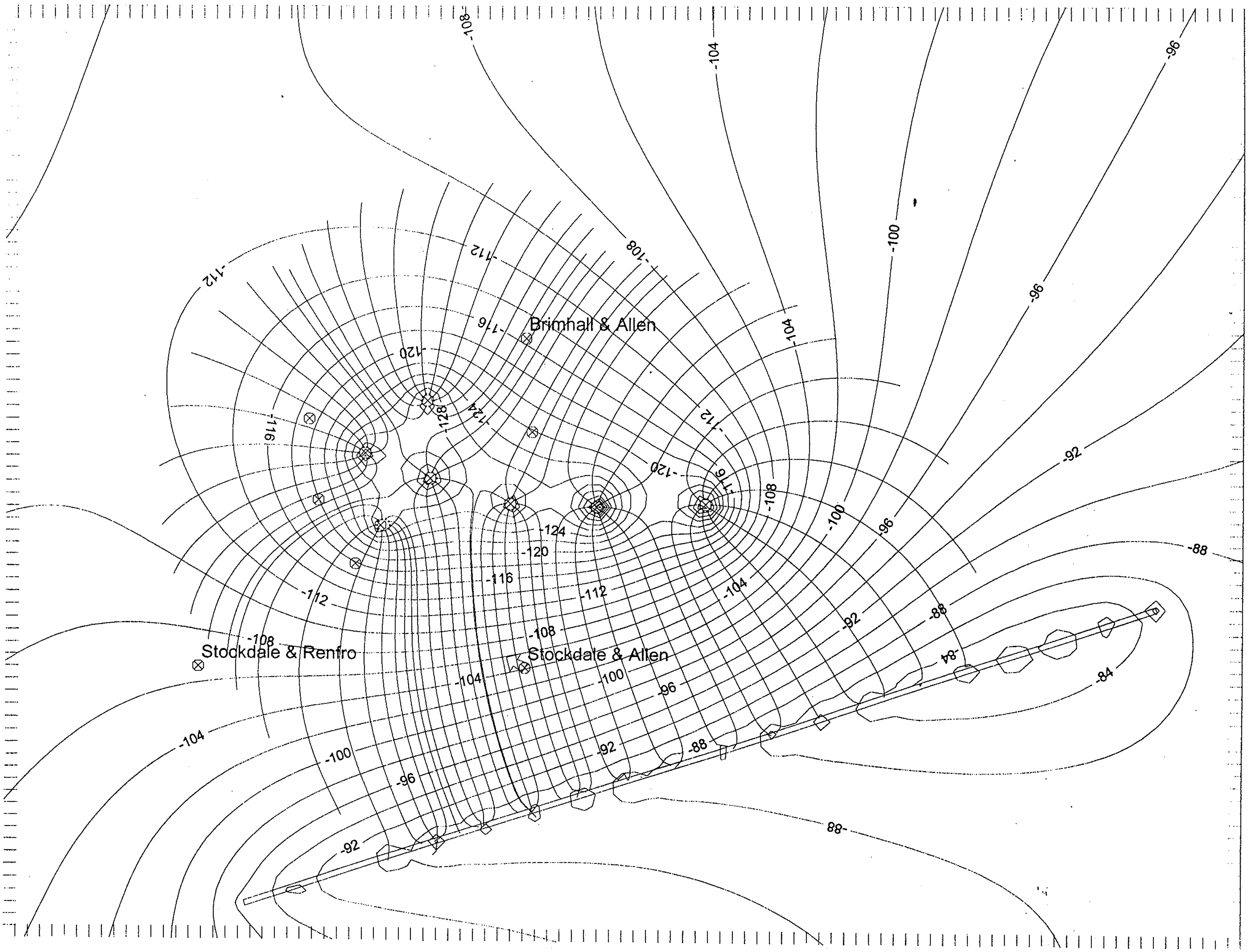
803

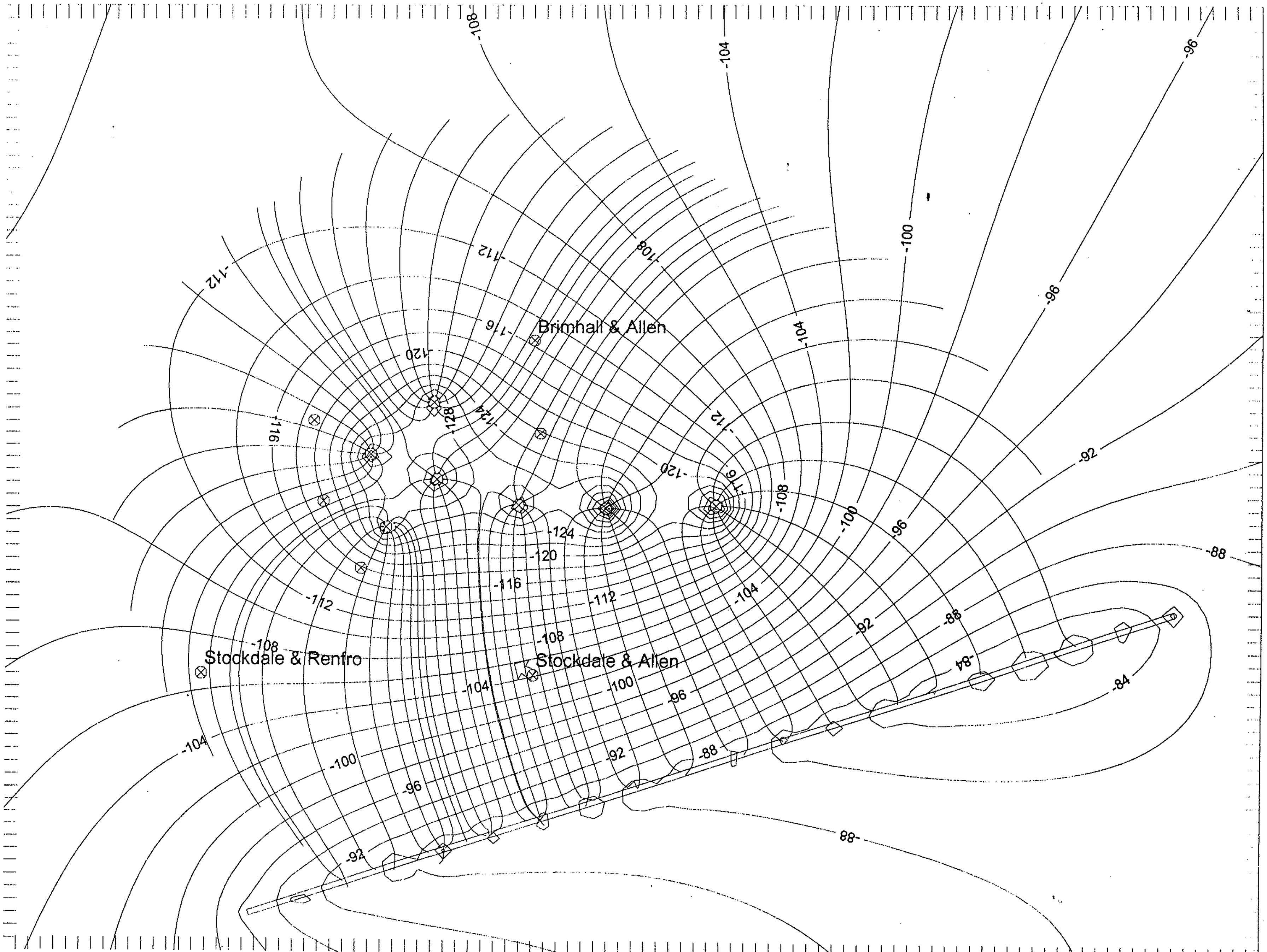


804

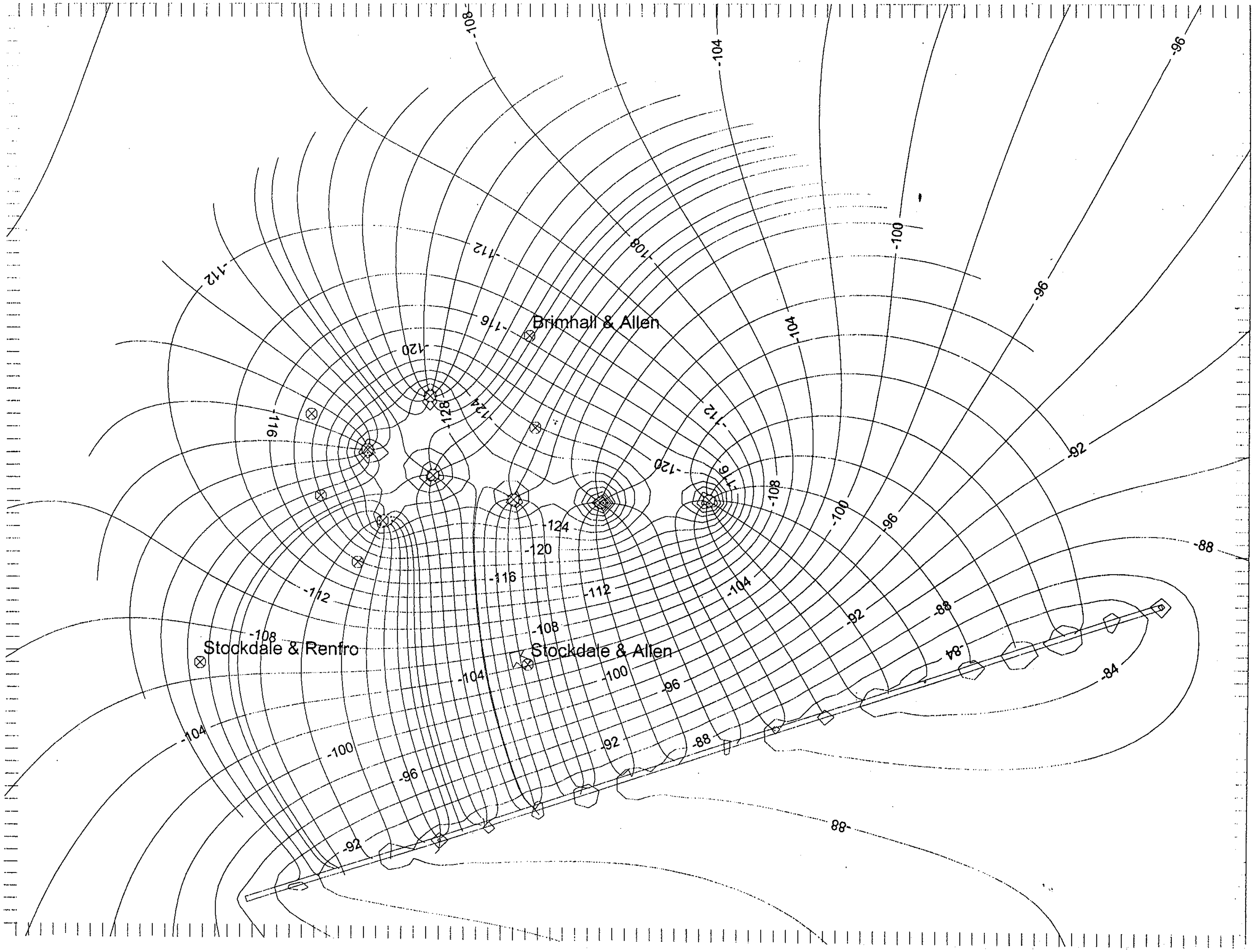


805


Brimhall & Allen
Stockdale & Allen
Stockdale & Renfro



Brimhall & Allen
Stockdale & Allen
Stockdale & Renfro
-84
-88
-92
-96
-100
-104
-108
-112
-116
-120
-124
-128



808

Summary of Other Analyses, Sets 9 - 10.

The modeling program can be used to analyze other factors and impacts beyond those required for the current scope of work. In set 9 we analyze the differences between the calculated output of the superposition method which SSS uses and the so-called centroid method which has been used by other workers on other projects. In set 10, we generate three hypothetical scenarios which show the impacts of local recharge ponds in addition to the impacts of recovery well pumping. The purpose of these hypothetical scenarios is to show the relative magnitudes of the various impacts in the project area and are not intended to represent any specific, actual operating scenario within this or any other scope of work.

Set 9. Comparison of the superposition method to the "centroid method".

SSS uses the method of superposition to calculate the combined drawdown from multiple pumping wells by calculating the drawdowns from each pumping well individually and then summing them up. SSS disagrees with the use of what is locally referred to as the "centroid method" for the determination of multi-well drawdowns. Instead of adding up the individually calculated drawdowns, the proponents of the centroid method calculate a single drawdown by adding up the individual pumping flow rates and treating the total as a fictitious single well located at the center of the well field. This "weighted average" fictitious well of the centroid method does not correctly compensate for the unique distribution of flow rates and distances of the actual well field, nor does it provide any computational savings in the math. On a theoretical basis, the values of "Q" cannot be added, and the values of "r" cannot be averaged in the logarithmic term of the flow equation and still get the correct result. We recommend that ID4 consider not using any results which may have been based on the centroid method.

We have presented a set of maps which illustrate the differences between the calculated results of the superposition method and the centroid method for the same wells and the same parameters. There is one drawdown map for each method and a third map which shows the numerical difference between the two methods. The difference map represents the error in the centroid method. The positive differences represent locations where the centroid method *underestimates* the true drawdown (particularly the real drawdowns at pumping wells and the ends of the well field) and the negative differences represent locations where the centroid method *overestimates* the true drawdown (particularly the fictitious drawdowns at the centroid and on the sides of the well field). The two zero lines separate the quadrants of too- high and too- low drawdowns and they represent the entire locus of points where the centroid method happens to make the correct prediction.

There is also a significant and perhaps even more important difference between the results of the two methods and that is the difference between the capture zones predicted by the two methods. We present drawdown and particle trajectory maps for t= 300 and 720 days for both methods in the absence of a groundwater gradient and in the presence of a groundwater gradient. The shape and extent of the capture zone calculated with the centroid method is very different than that of the standard method, and the centroid method gives particle trajectories that are completely inaccurate for useful capture analysis.

Set 10. Superposition of hypothetical recharge impacts on hypothetical pumping impacts.

SSS has created three hypothetical recharge scenarios strictly for purposes of illustration. The first scenario is for continuous recharge in just the RRB ponds west of Allen Rd superimposed on river recharge and the regional gradient but without any wells pumping. The second scenario is the same as the first scenario except that the wells are pumped simultaneously for 300 days. The third scenario is the same as the second scenario except that two small recharge ponds have been added to the east of Allen Rd.

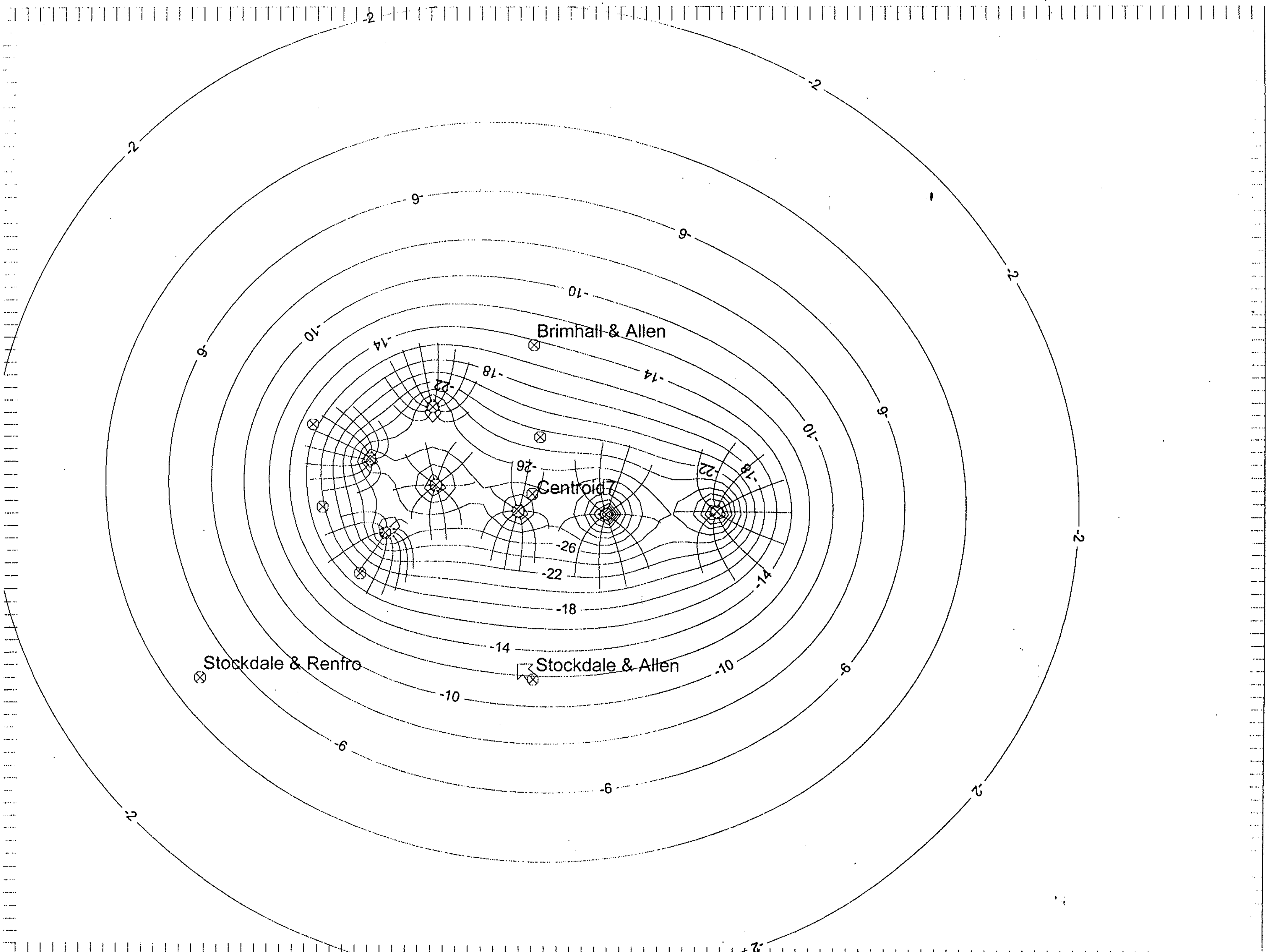
Appendix 6.

SET 09.

ID4 / KT / RRB Well Field Model Parameters.
Set No. 9.

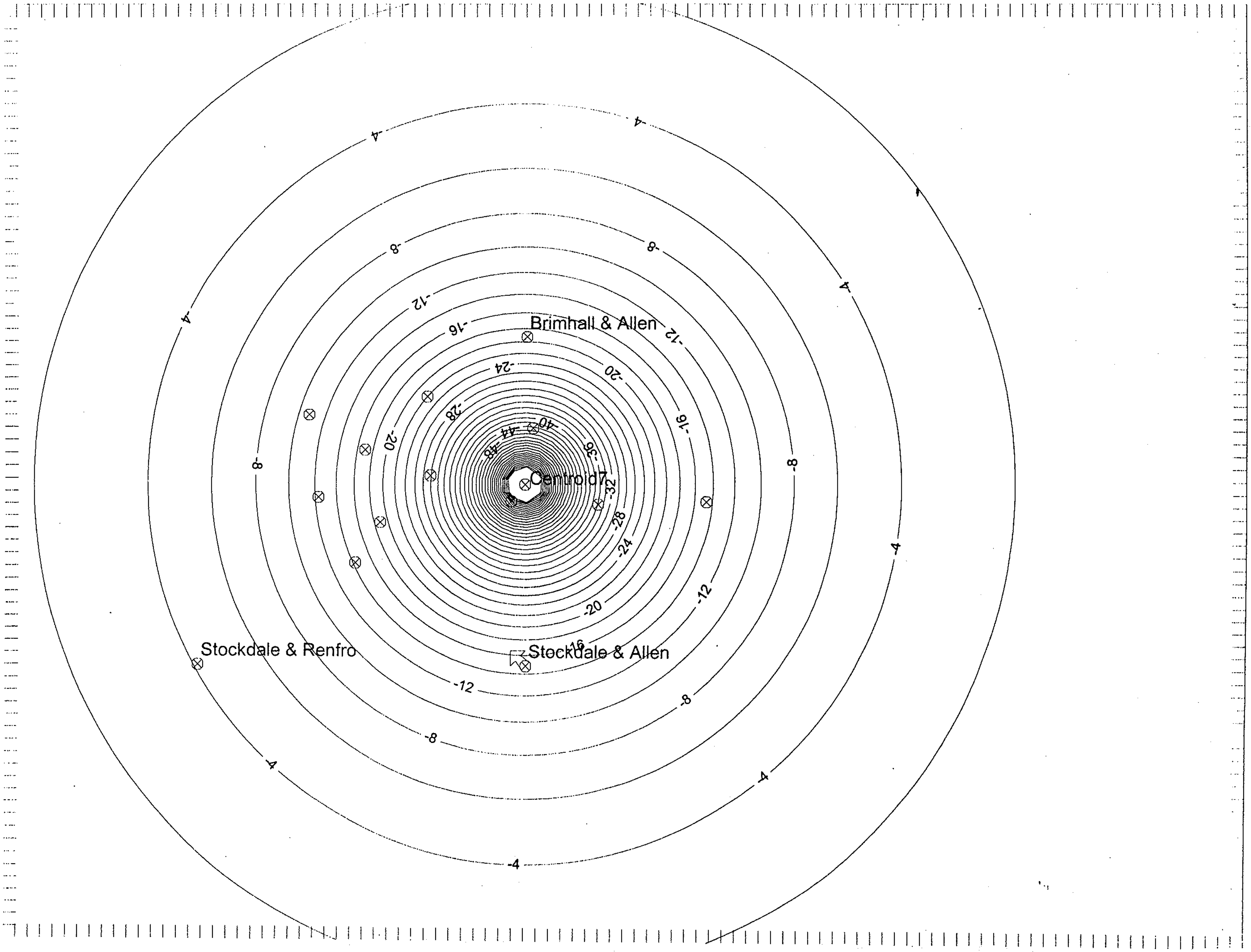
Note: only CHANGES are tabulated between Runs, from left to right.
Note: once made, CHANGES persist until modifed again.

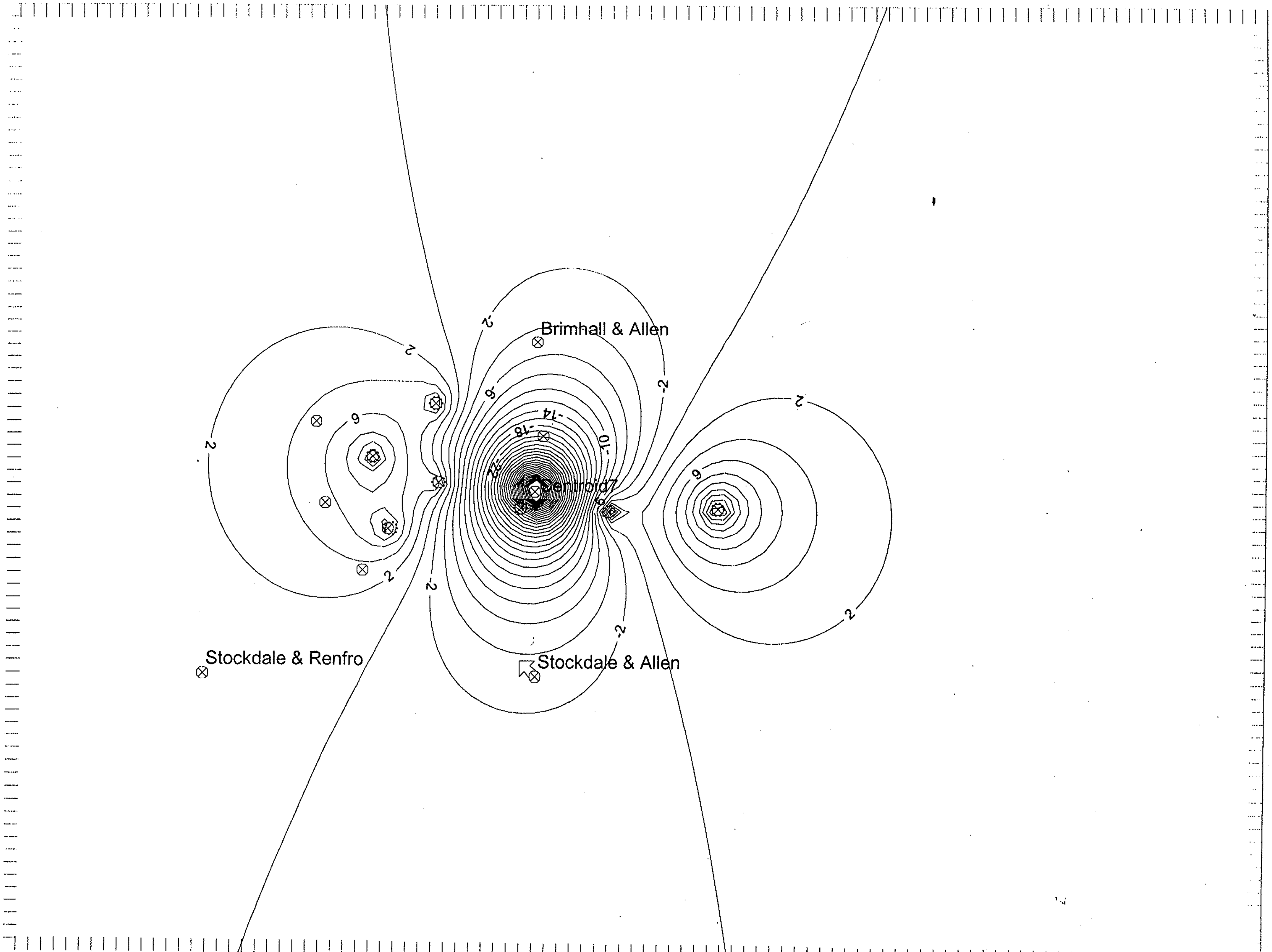
		base case												
model		leaky	leaky	leaky	leaky	leaky	leaky	leaky						
calc		transient	transient	transient	transient	transient	transient	transient						
Run No.		ref.only	901	902	903	904	905	906						
					(diff 1)			(diff 2)						
H.Cond.	K	80												
Btm	B	-450												
Top	T	-200												
Ref Hd	R	0				-100		0						
Grad	G	0				0.001								
Rchg	Rch	0												
Por	P	0.3												
Stor	S	5.60E-04												
Lkg	L	3200												
pump-t	t	300												
Ref Hd	Ref	0				-100		0						
Grad	G	0				0.001								
Azim	A	135				150								
Ref-X	X	0												
Ref-Y	Y	0												
Const Hd	CH	no												
trace-t	tt	300												
Flow Rate	Q	full		0	neg. full		0							
Wells	W	all		none	all		none							
River Rch	Q	0				-210	-210							
Centroid7	Q	0		3888000		0	3888000							



Brimhall & Allen
Centroid7
Stockdale & Allen
Stockdale & Renfro
-2
-6
-10
-14
-18
-22
-26

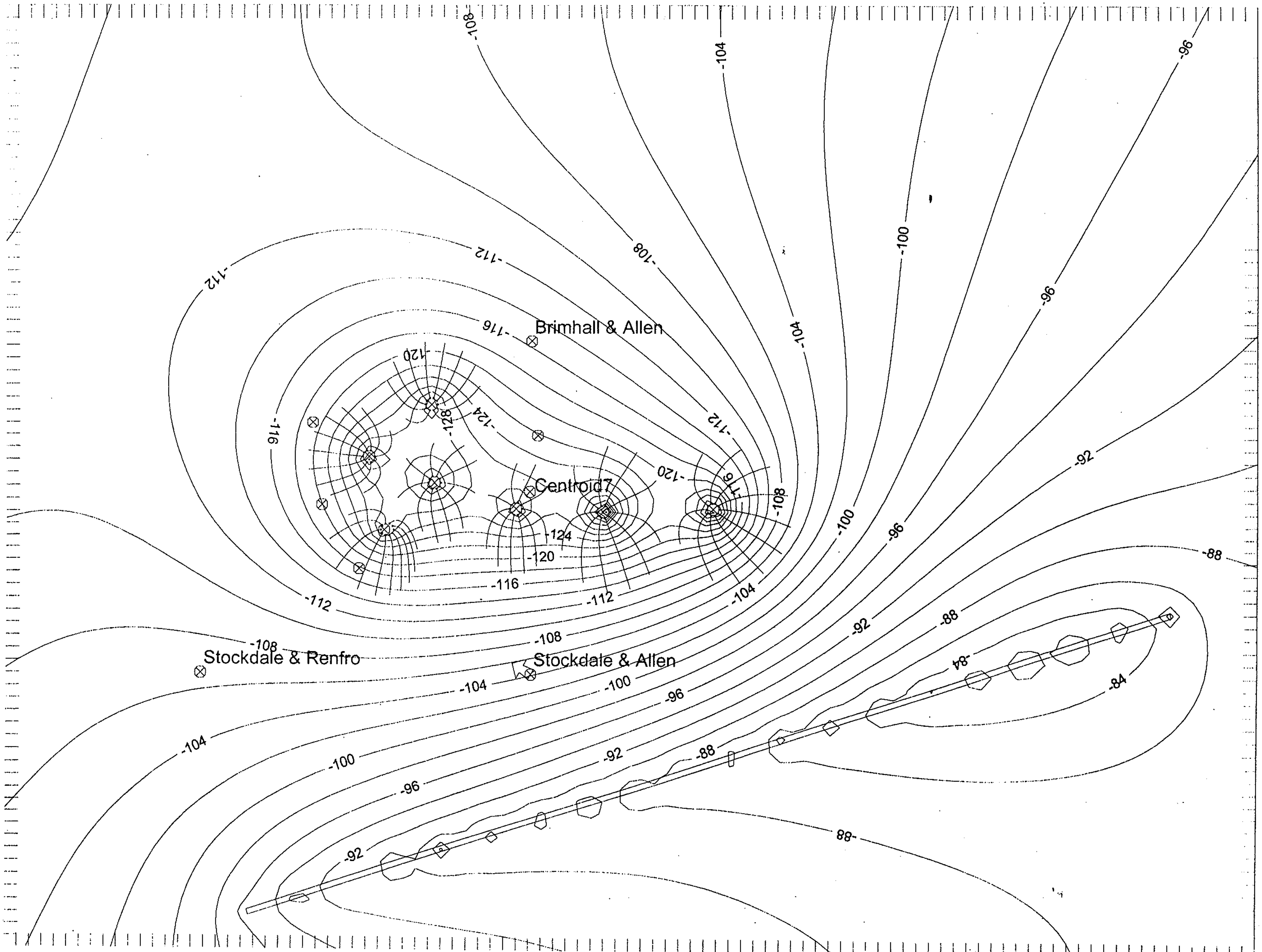
196


Brimhall & Allen
Centroid7
Stockdale & Allen
Stockdale & Renfro
-4
-8
-12
-16
-20
-24
-28
-32
-36
-40
-44
-48

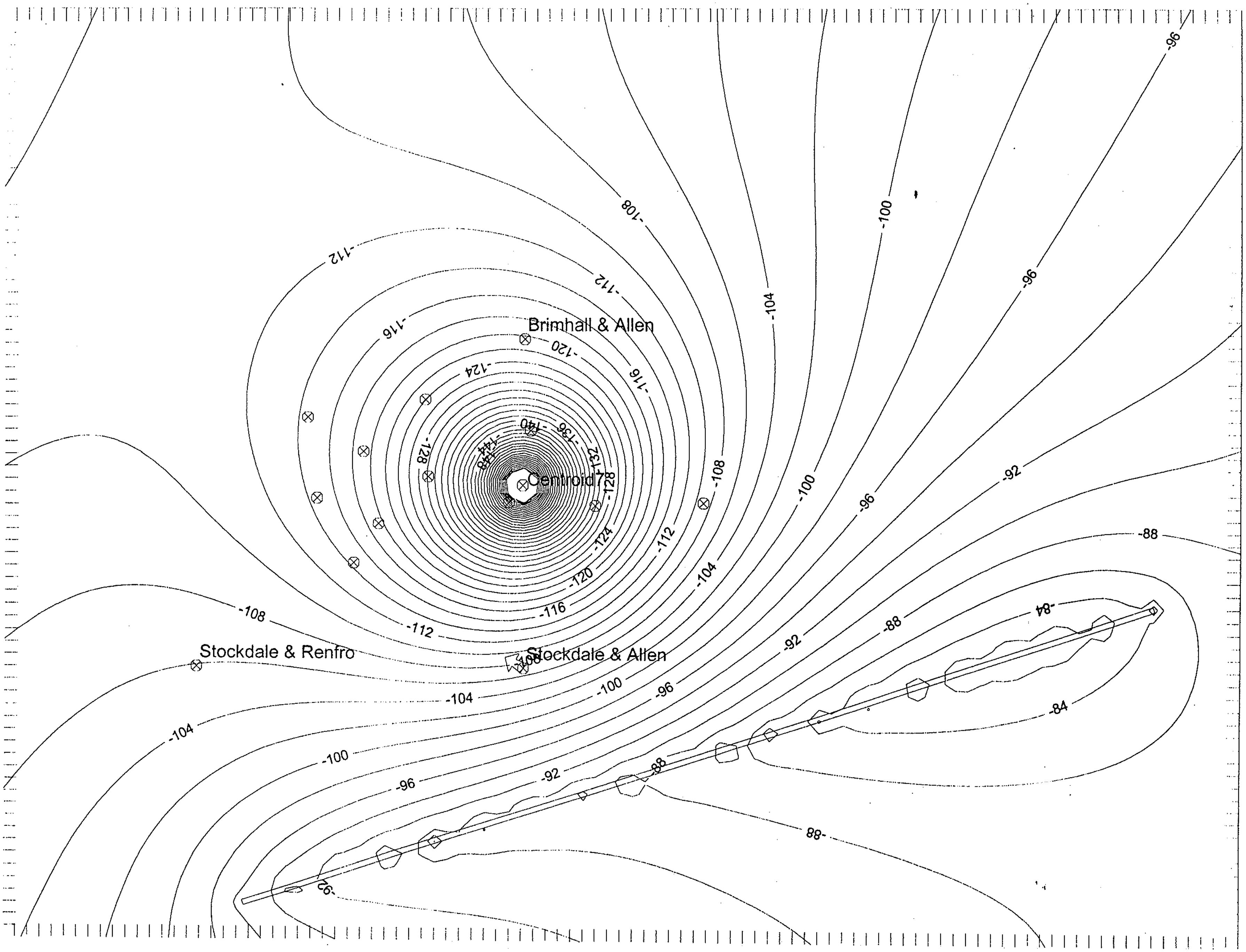


Brimhall & Allen
Centroid7
Stockdale & Allen
Stockdale & Renfro

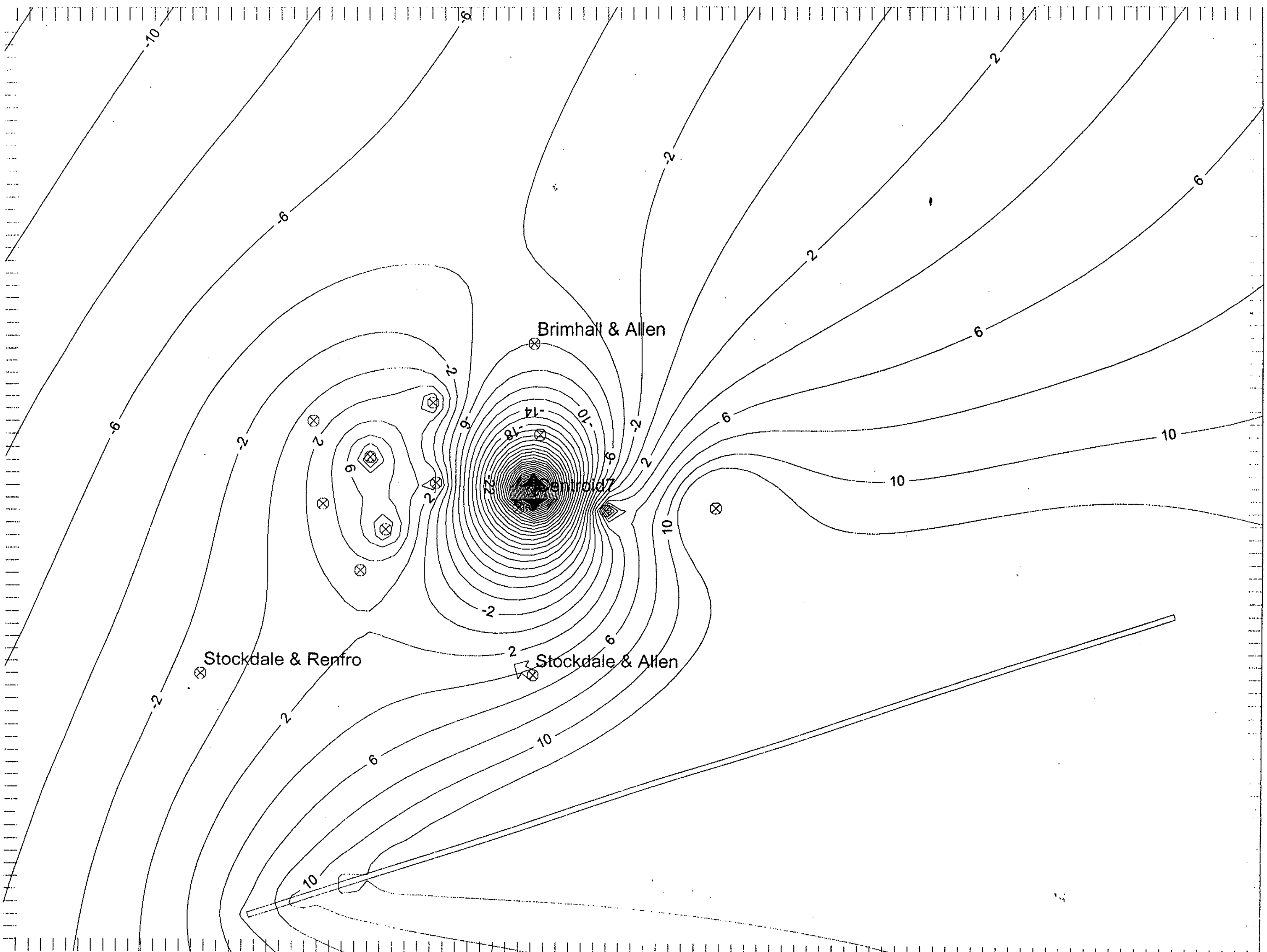
903


Brimhall & Allen
Centroid7
Stockdale & Allen
Stockdale & Renfro
-84
-88
-92
-96
-100
-104
-108
-112
-116
-120
-124
-128

904


Brimhall & Allen
Centroid7
Stockdale & Allen
Stockdale & Renfro

90.5


Brimhall & Allen
Centroid7
Stockdale & Allen
Stockdale & Renfro

Appendix 6.

SET 10.

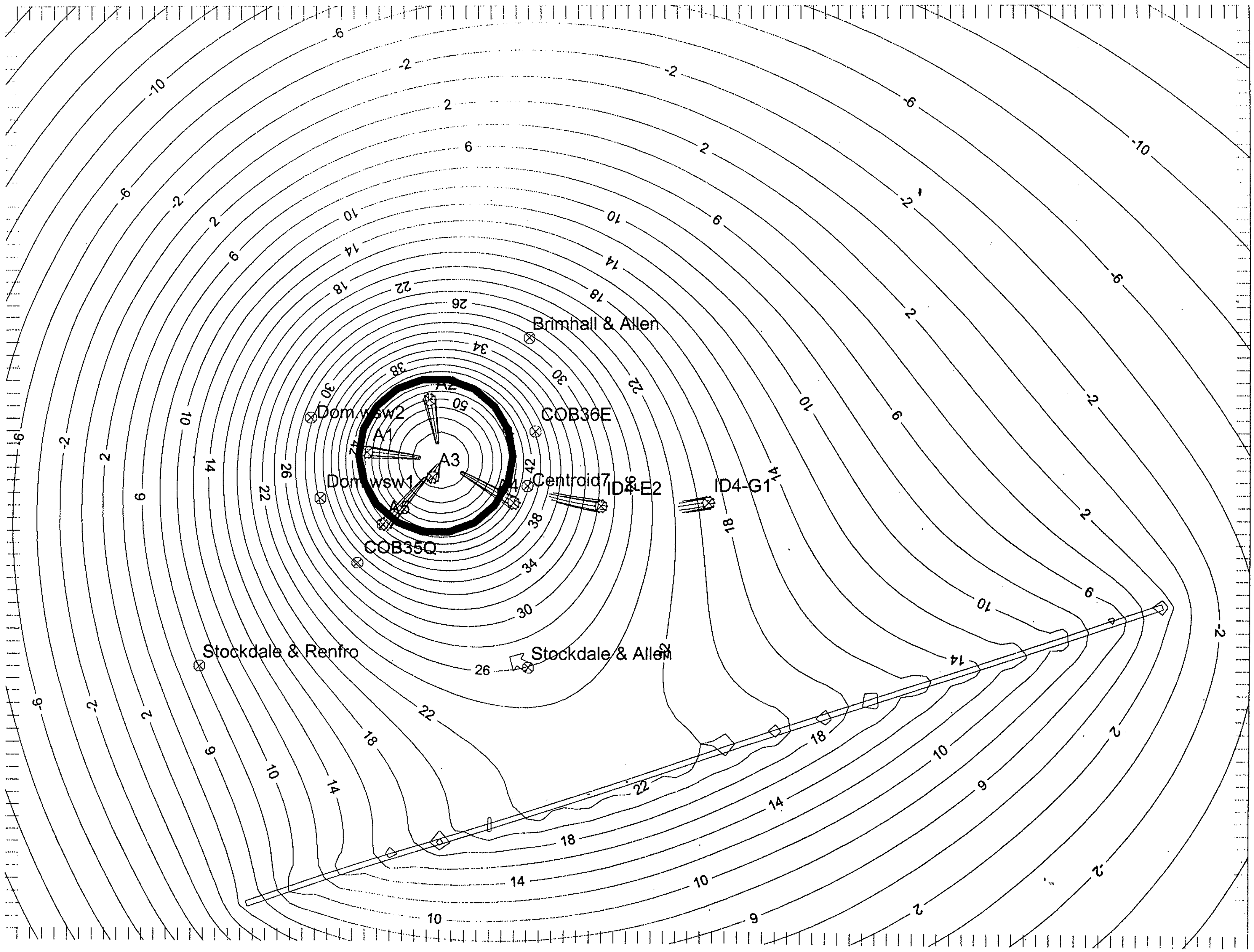
ID4 / KT / RRB Well Field Model Parameters.

Set No. 910.

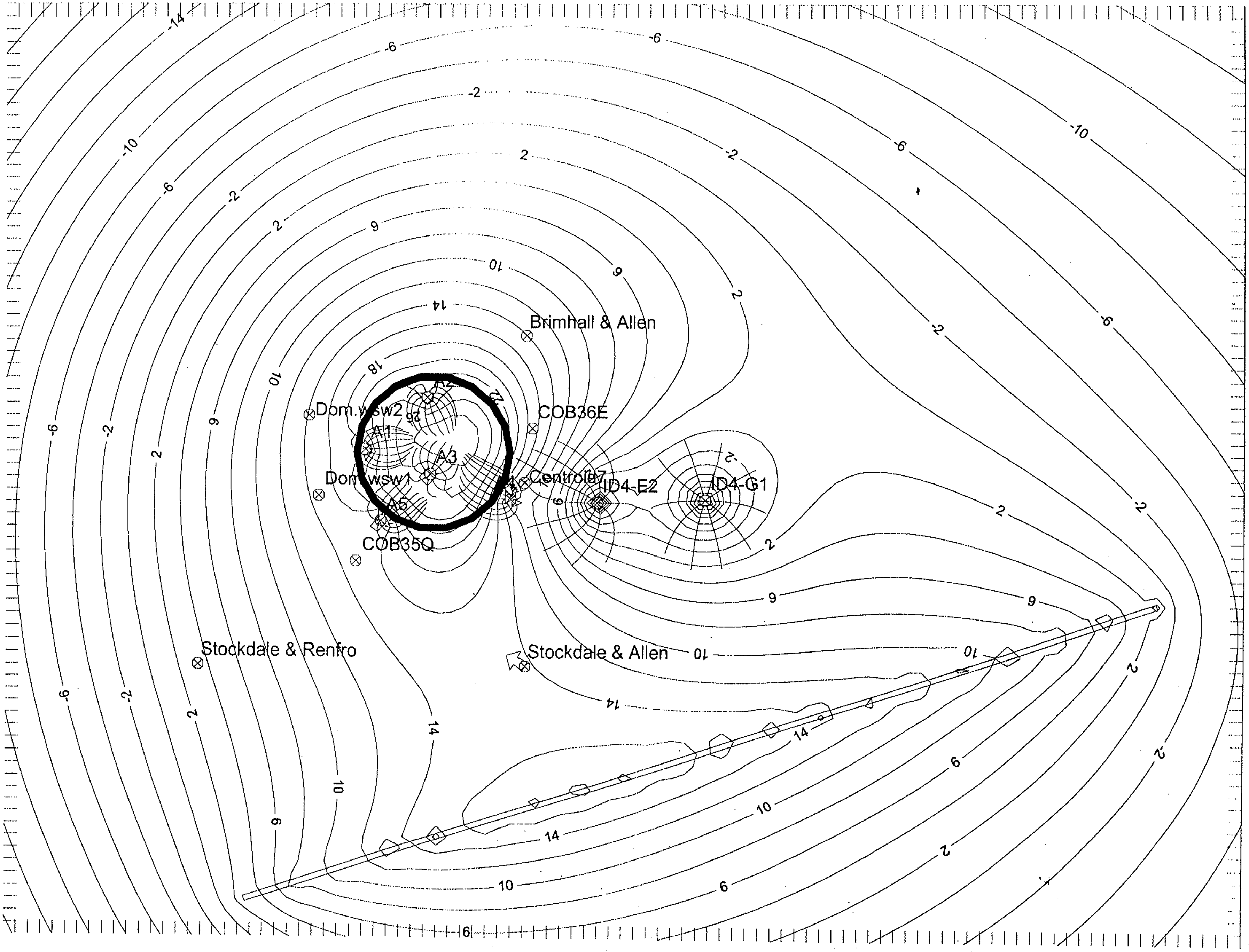
Note: only CHANGES are tabulated between Runs, from left to right.
Note: once made, CHANGES persist until modifed again.

		base case												
model		leaky	leaky	leaky	leaky	leaky	leaky	leaky				leaky	leaky	leaky
calc		transient	transient	transient	transient	transient	transient	transient				transient	transient	transient
Run No.		ref.only	901	902	903	904	905	906				1001	1002	1003
					(diff 1)			(diff 2)						
H.Cond.	K	80												
Btm	B	-450												
Top	T	-200												
Ref Hd	R	0				-100		0				-100		
Grad	G	0				0.001						0.001		
Rchg	Rch	0												
Por	P	0.3												
Stor	S	5.60E-04												
Lkg	L	3200												
pump-t	t	300												
Ref Hd	Ref	0				-100		0				-100		
Grad	G	0				0.001						0.001		
Azim	A	135				150						150		
Ref-X	X	0												
Ref-Y	Y	0												
Const Hd	CH	no												
trace-t	tt	300												
Flow Rate	Q	full		0	neg. full		0					0	full	full
Wells	W	all		none	all		none					none	all	all
River Rch	Q	0				-210	-210					-210	-210	-210
Centroid7	Q	0		3888000		0	3888000							
Pond Rch	Q	0										pond 1		ponds 1-3

pond 1: 110 ac @ 1 ft/d = 4.71xE06 cf/d; r = 1235 ft, cen @ (-1500, 3400)
pond 2: 4.15 ac @ 1 ft/d = 181xE03 cf/d; r = 240 ft, cen @ (400, 2000)
pond 3: 2.88 ac @ 1 ft/d = 126xE03 cf/d; r = 200 ft, cen @ (2200, 2400)

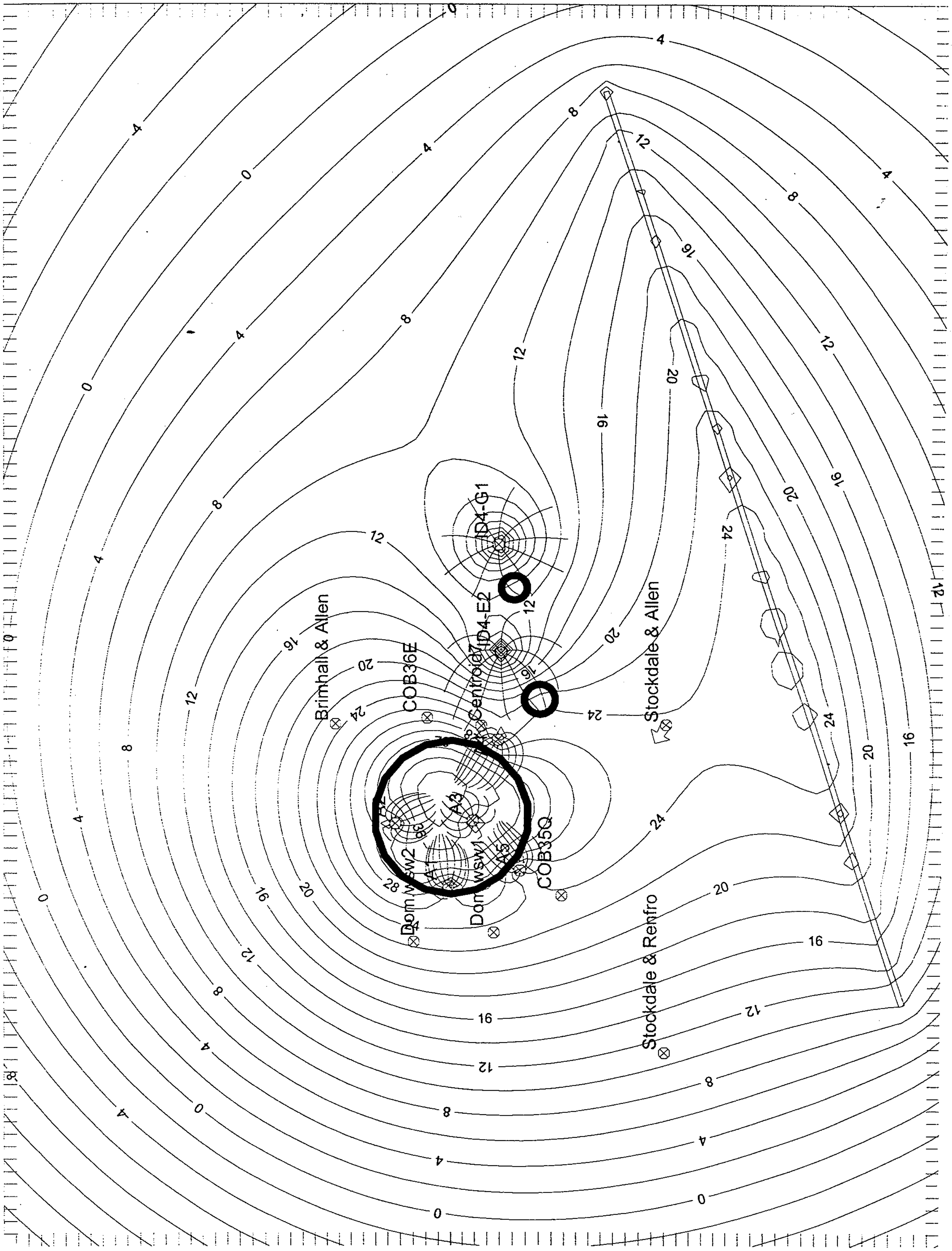

Brimhall & Allen
COB36E
Dom.wsw2
A1
A2
A3
Dom.wsw1
COB35Q
Centroid7
ID4-E2
ID4-G1
Stockdale & Renfro
Stockdale & Allen

1001



Brimhall & Allen
COB36E
Dom.wsw2
A1
A2
A3
A5
Dom.wsw1
Centroid7
ID4-E2
ID4-G1
COB35Q
Stockdale & Renfro
Stockdale & Allen

1002



Brimhall & Allen
COB36E
Centroid7
ID4-E2
ID4-G1
Stockdale & Allen
COB35Q
Stockdale & Renfro
A3
A5

1003

Appendix 7.

Limitations of the Analyses.

SSS has evaluated several sets of base case and non- base case aquifer conditions to determine the predicted impacts of the proposed pumping program. The uncertainties in the calculated results are due to several factors which we briefly summarize in this Appendix.

<u>Non - project wells.</u>

There are three issues related to the impact of non- project wells in the local area. The first issue is the effect of water table decline due to the pumping of these non- project wells which is in addition to, and superimposed upon, the drawdown caused by the project wells. We have not included any hypothetical scenarios which takes this into consideration.

The second issue is that these non- project wells are removing water from aquifer storage which is not included in the project water balance. Even if the project remains in balance, the local area may still suffer a net shortage of recharge which will create a net decline in water levels, which will ultimately change the aquifer behavior from semi-confined to unconfined. We have already included this hypothetical scenario in our general analysis, and it is important to recognize the potential for shallow aquifer dewatering by pumping non-project wells.

The third issue is that non- project wells create capture zones of their own which extend outward into surrounding areas which are outside of the capture zone limit of just the project well field alone. It is possible that these surrounding wells draw contamination into the project area that would not have arrived here otherwise. Such a capture analysis is outside the scope of this analysis. While there are limits to the possible magnitude of this potential impact, the wells of greatest potential concern would be wells which are close to the project well field and those which are to the east of the well field.

<u>Changes in the groundwater gradient.</u>

Based on KCWA groundwater elevation maps for the area, we have observed an overall change in the groundwater gradient as the climate swings from wet to dry conditions. During a wet cycle, the recharge in the three- mile stretch of the Kern River channel from Allen Rd east to Coffee Rd tends to create a northwesterly component to the overall gradient on the north flank of the river recharge axis (for example, see KCWA maps for 1996 - 1998). During a dry cycle, the absence of recharge in this stretch causes a westerly gradient to dominate due to the effects of aquifer dynamics farther to the east (for example, see KCWA maps for 1991 - 1993).

This shift may cause contaminant plumes located outside of, but close to, the long term capture zone limit to move into the capture zone. The reverse is not really possible, i.e., contaminant plumes leaving the capture zone, because even though particle trajectories say it is possible, actual contaminant migration invariably leaves in situ residues behind in its pathway which linger as continuing in situ sources of low- grade contamination for many years thereafter. We have included the uncertainty in ground water gradient in our analysis by using a long- term background average ground water gradient behavior within the computational model, based on the observed trends from the KCWA historical ground water elevation maps.

Interzonal water quality changes.

Aquifer storage and recovery (ASR) projects in the Kern Fan area generate large and persistent vertical ground water gradients which cause interzonal flow. This flow causes significant interzonal mixing which may be of concern if project pumping causes undesirable impacts on zones which previously had acceptable water quality. We have not included the vertical flow component in our capture zone models or our qualitative comments on the potential for contaminant capture.

In our opinion, which is based on our interpretation of some of the available geochemical data for another project, both the general mineral chemistry and the constituent- of- concern (COC) chemistry varies significantly between some depth zones within the aquifer. In our opinion, this trend is being obscured by the use of blended- water analyses from multi- zonal water wells (often by choice in data- poor areas) and by ignoring the depth dimension in the bulls- eye approach to water quality mapping.

We do not currently have enough baseline water quality data to predict the quantitative water quality changes in the project area due to project pumping. There is a general opinion that the shallow unconfined aquifer has the poorest water quality and that the water quality improves with depth. Such statements have little meaning unless they are qualified as to the water quality criteria of interest, whether it be total dissolved solids, hardness, or some constituent of concern. Some generalizations may help. Good quality water, by local standards, refers to Kern River water which has a low TDS = 150 mg/l, acceptably low concentrations of naturally- occurring objectionable constituents (such as arsenic at 4 ug/l), and no manmade constituents at concentrations of concern, if detectable at all (such as nitrate at _).

In general terms, the groundwater in the shallowest zones in the project area has a constituent chemistry which most- closely reflects the chemistry of the local recharge water combined with shallow local infiltration. The deeper zones increasingly reflect the chemistry of more- distant recharge which travels different and longer flowpaths before arriving under the project area. As long as ground water extraction remains less than the local yield of the aquifer, the differences in water chemistry in different depths zone will persist. But as ASR projects are increasingly operated in the area, the primary direction of recharge will change from lateral flow in the different zones to downward vertical flow from the shallower layers. The recovery pumping will accelerate the downward leaky recharge to the aquifer and the consequent dewatering of the shallow zone will accelerate the lateral inflow of shallow water into the project area. This change in local aquifer dynamics will cause an interzonal blending in which the water chemistry in successively deeper layers will become more like that of the near- surface recharge.

To the extent that this downward flux transports poorer quality water into deeper zones, it represents a water quality degradation of the deeper water. However, it is not clear that all of the objectionable constituents are isolated at the top, so a blending of this type may actually serve to dilute the concentration of certain deeper COCs, perhaps such as arsenic and/or uranium, and/or H_2S gas at depth, with water of overall less objectionable qualities. And it will probably be found that the flux of poorer quality shallow water through the silty aquitards will be cleaned up by the natural filtration effect of the finer- grained sediments.

All of this being said, there is less to be done at this time than there is to be learned by paying careful attention to the effects of this aquifer behavior during the project operation. We recommend that the design of a groundwater monitoring program include multi-zone WQ monitoring in what are identified to be key locations.

Uncertainty in predictive modeling.

There are several causes of uncertainty in the outcome of a predictive forecast and it is useful to keep the relative importance of these causes in perspective.

Natural variability. The single most significant cause of uncertainty is natural variability, i.e., the complexity, heterogeneity, and randomness in the real world which are impossible to fully identify or evaluate at relevant scales of measure. In this project, we know that the aquifer is more complex in ways which we may or may not recognize but can't model because of insufficient data. For example, we know that the silty layers seen in the E-log of one well rarely correlate with the silty layers seen in adjacent wells. But we can't model all of these individual layers because we don't actually know where they start and end in the unobserved spaces between wells. The same is true for boundaries which are there but have not yet been detected by the existing investigations.

We must therefore try to represent the known or suspected complexity with a simpler component within our model which best approximates the expected behavior of the real earth by lumping the complex properties together in the form of a simpler analog. The practice of "lumped parameter" modeling is a simplification of choice as well as necessity. Even if it were possible to represent every sand grain and every pore space in the aquifer, the increase in microscopically detailed complexity may not contribute anything to improve the accuracy or reliability of the results. It is one of the hard-won skills of good modeling to know when and where a simpler approximation will be an effective and accurate representation of the real system.

A corollary effect of natural variability is that the true aquifer parameters will always be somewhat different than those in the model at some place or at some time. Even if we could precisely determine the true average value for every parameter, those local parts of the aquifer which are higher or lower than the average value and have observation wells located in them, will be observed to behave differently than predicted by the model. Since predictive modeling is often used *before* projects have begun, it is often true that a sufficient amount of good data doesn't even exist to estimate the average properties of the aquifer let alone map the full range of variability at all locations. Often a sufficiency of data doesn't exist until such a projected has operated for many years. So, when comparing a predicted behavior to a subsequently observed behavior, it would be a mistake to treat point- by- point differences as a parameter error when those differences can be adequately explained as being caused by simple, undeterminable, natural variability about an average value.

Another effect of natural variability applies to the inability to predict future naturally-occurring or manmade events or behaviors, in addition to the variability in physical properties. For example, highly variable weather conditions can deviate significantly from average behavior without being considered anomalous, so that any *particular* predicted event has a significant chance of being different than the actual occurrence even though the prediction is a "correct" one. For these types of conditions, the correct prediction is actually a set of predictions covering the full range of possible values, with a probability of occurrence attached to each one. So in this project, we predict aquifer drawdowns due to pumping and our model stipulates that the actual future drawdown behavior will be controlled in part by the amount and timing of recharge which is controlled by the climatic weather cycle. So, we have identified a range of possible drawdown scenarios based on two possible weather- controlled recharge scenarios, i.e., sufficient recharge and insufficient recharge.

Judgment. The second significant cause of uncertainty is errors in judgment by the modeler, including such mistakes as selecting an inapplicable model or poor model parameters, doing the work incorrectly, or failing to recognize and correct "catchable" mistakes. These errors in judgment range from making an informed choice under difficult conditions or with very little data to blatant mistakes. There is probably little chance of a non-expert catching errors in judgment other than, perhaps, blatant mistakes.

In our opinion, there are two ways to try to catch judgment errors. The first is to get a second opinion from a qualified expert. The second is to take the time to learn enough basics to make a critical review of the work. After all, the accuracy of your own work may depend on these results. And then, require clear, complete, and verifiable documentation beyond simple numerical QA/QC with any modeling project and simply evaluate the work product for logic, consistency, clarity, and credibility.

Expectation. A third cause of uncertainty is errors of expectation on the part of an inexperienced modeler or the final user of the predictive output. Errors of expectation can include expecting too much and expecting too little. Unreasonably high expectations often come from a lack of understanding of the issues of natural variability. Examples of such errors of expectation include the assumption that there is only a single possible answer or that it is single- valued; that the answer is precise and accurate and, if correct, will be verified to a high degree by the actual observed outcomes; that the answer must be right because modeling is a numerical procedure and computational accuracy is mistaken as being the same as representational accuracy; or that the modeling procedure is wrong or useless or that mistakes must have been made if the predicted results and actual results disagree in some way.

Low expectations often come from a lack of understanding of how powerful and sophisticated predictive modeling can be in the hands of a competent expert. Many business people, policymakers, engineers, and consultants go about their particular business unaware that predictive modeling tools exist for almost every type of process or system including groundwater phenomena such as the flow behavior of rivers, water supply reliability, weather patterns, basin analysis, flow behaviors, and contaminant plume migration.

Unlike errors of judgment by a trained practitioner, errors of expectation are not a matter of right or wrong. Getting it wrong while learning what to expect is the normal process for all of us. The lesson is that if modeling is not part of one's expertise, then 1. hire an expert rather than trying to do it yourself, 2. talk to your expert about reasonable expectations, and 3. learn something about the required inputs, the process itself, and the form of the expected output so you can bring some critical review to the results.